VOLUME 84 NO. PO4
AUGUST 1958
PART 1

JOURNAL of the

Power Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

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2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

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8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class mail privileges are authorized at Ann Arbor, Michigan.

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

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ROCKFILL DAMS: CHERRY VALLEY CENTRAL CORE DAM[a]

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(Proc. Paper 1733)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

This paper describes the construction of Cherry Valley Dam of the Hetch Hetchy Water Supply of the City and County of San Francisco. Cherry Valley Dam is of the central core rockfill type. The central core is composed of compacted decomposed granite and the rockfill is of freshly quarried granite, placed in lifts of 15 to 30 feet and sluiced. The performance of the

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1733 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at meeting of ASCE, Portland, Ore., June, 1958.

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embankment during the first two fillings of the reservoir is also described and measurements of the settlements and deflections are given.

Embankment Design

The design of the Cherry River Project of the City and County of San Francisco has been presented in detail in Convention Preprint Paper No. 71, "The Cherry River Project of the City of San Francisco" by M. L. Dickinson, M. ASCE. (March, 1953). For the benefit of those who do not have access to this paper the design of the dam embankment will be reviewed briefly.

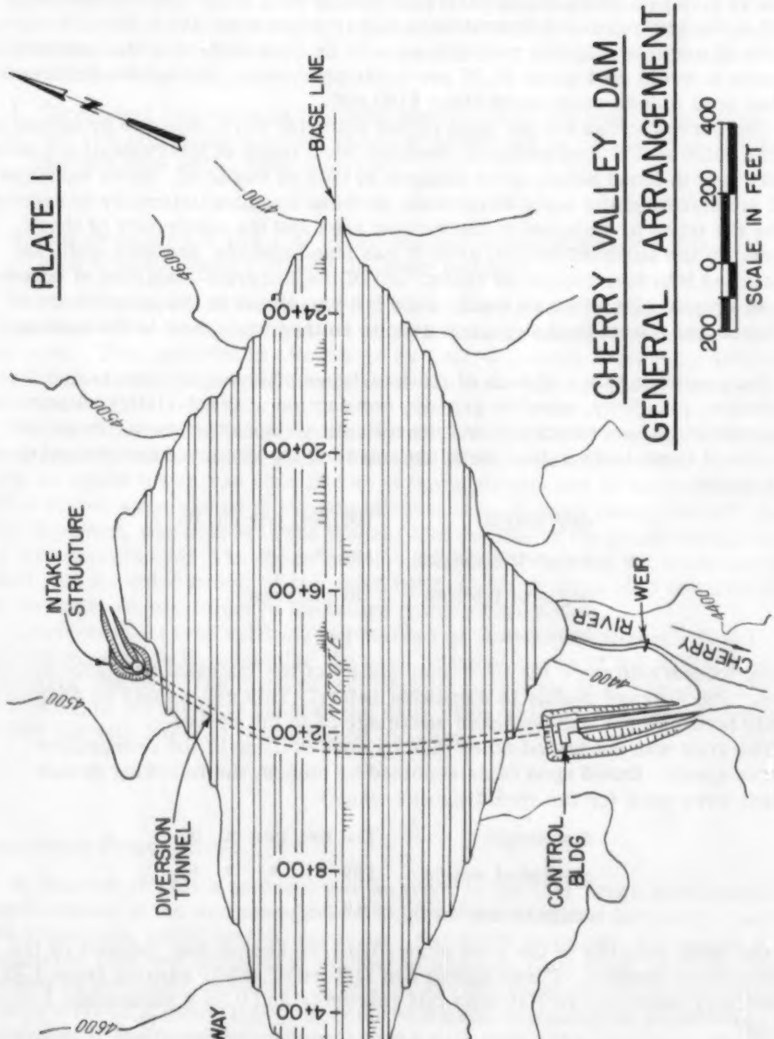
Cherry Valley Dam is of the composite earth and rock-fill type, with the impervious core centrally located in the cross-section. A two layer transition zone, 20 feet in total thickness, is placed between the impervious core and the rock envelope, both upstream and downstream. The dimensions are as follows:

Crest length, feet	2,600
Crest elevation, feet	4,715
Crest height above streambed, feet	315
Depth streambed to bedrock, feet	15
Height, total, feet	330
Crest width, feet	40
Thickness at base, maximum, feet	1,320
Volume, cubic yards	
impervious core	2,932,000
transition zones	593,000
rock envelopes	3,475,000
Total	7,000,000

This type of dam was selected in order to make the best possible use of local materials. Ample deposits of material for the impervious core were found within two miles of the damsite, and excellent granite for the rock envelopes was obtained from a quarry on the right abutment, just downstream of the dam. Material for the transition zones was obtained from deposits in the reservoir area. The embankment section was proportioned to use as much core material as possible while still maintaining the desired factors of safety under the various conditions of loading. A core with the required degree of impermeability could have been obtained with a thinner section, but since the cost per cubic yard of the rock fill was about twice that of the core, it was desirable to use a section containing the maximum proportion of core. However, if the embankment were to be built entirely of the core material, the slopes of the upstream and downstream faces of the embankment would have to be flattened to maintain the required stability of the section. This would increase the total volume of material required for the embankment and also the length of the combined outlet and diversion tunnel. In the case of Cherry Valley Dam the total cost of the embankment would remain about the same since the increase in volume would be offset by the lower unit cost of the material, but the cost of the tunnels would be increased about 50% due to the greater length required.

The outside slopes of the rock fill were made as steep as possible while maintaining the required stability. The overall slope of the outside face of the rock is 2 horizontal to 1 vertical. The upstream slope of the impervious

PLATE I



CHERRY VALLEY DAM
GENERAL ARRANGEMENT

core is 0.70 to 1 while the downstream face is 0.75 to 1. The difference of 0.05 in the upstream and downstream slopes represents about 160,000 cubic yards of material. If this material were to be rock instead of decomposed granite it would cost about $1.00 per cubic yard more, so that the difference in the core slopes represents about $160,000.

The borrow areas for the impervious material were explored by means of auger holes at 250 foot centers. Samples were taken at intervals of not over seven feet in these holes, or at changes in type of material. Sieve analyses and moisture density tests were made on these samples primarily to determine the types of material in the borrow area and the uniformity of these types. In the selected borrow area it was found that the material could be classified into four groups or types. All of the material consisted of decomposed granite classified as a silty sand but variations in the proportions of feldspar and quartz in the original granite caused variations in the residual material.

Composite samples of each of the four types of material were tested for gradation, plasticity, specific gravity, compaction characteristics, direct shear strength, permeability, and consolidation characteristics. From the results of these tests values were assumed for the design of the embankment, as follows:

dry weight	106 lbs. per cu. ft.
saturated weight	131 " " " "
angle of friction	27 degrees
cohesion	0

These values represent the minimum quality of the material available for the core. The average quality is somewhat better. This represents an additional safety factor which was not taken advantage of in the design.

The rock was not tested other than by core drilling of the prospective quarry areas. Based upon tests reported by others, the following design values were used for the rock fill:

dry weight	110 lbs. per cu. ft.
saturated weight	132 " " " "
angle of friction	45 degrees

The local stability of the rock slopes between berms was checked by the infinite slope method. These slopes had factors of safety ranging from 1.33 with empty reservoir to 1.10 with full reservoir and 0.05 g earthquake forces acting.

The overall embankment was checked by use of the slip circle method. The minimum factors of safety for the various design conditions are:

Normal operating conditions	1.50
Infrequent operatiing conditions such as sudden drawdown	1.25
Normal operating conditions plus 0.05 g earthquake	1.10

The transition or filter zone between the core and the rock consists of two layers or zones. The finer layer, adjacent to the core, is 8 feet thick while the coraser layer is 12 feet thick. The gradation of the material in these zones was based on the criteria that:

1. To prevent migration of the finer material into the coarser material the 15% size of the coarser material should not be greater than 5 times the 85% size of the finer material;

and

2. For drainage the 15% size of the coraser material should not be less than 5 times the 15% size of the finer material.

As an added protection against migration of the fine material into the rockfill, selected fine material from the quarry was placed adjacent to the transition zone. This material in effect acts as a third transition zone and actually reduced the cost of the embankment since the material would otherwise have been wasted.

During construction the river was diverted through a 16 foot diameter concrete lined tunnel under the right abutment. Upon completion of the embankment an intake tower was constructed at the upstream end of the tunnel and outlet valves were placed at the downstream end. A steel lining, 15'-4" inside diameter, was placed in the tunnel from the line of the grout curtain to the downstream end. The space between the steel lining and the concrete was filled with a sanded grout. A tee and a butterfly valve were also installed at the downstream end to serve the future power development.

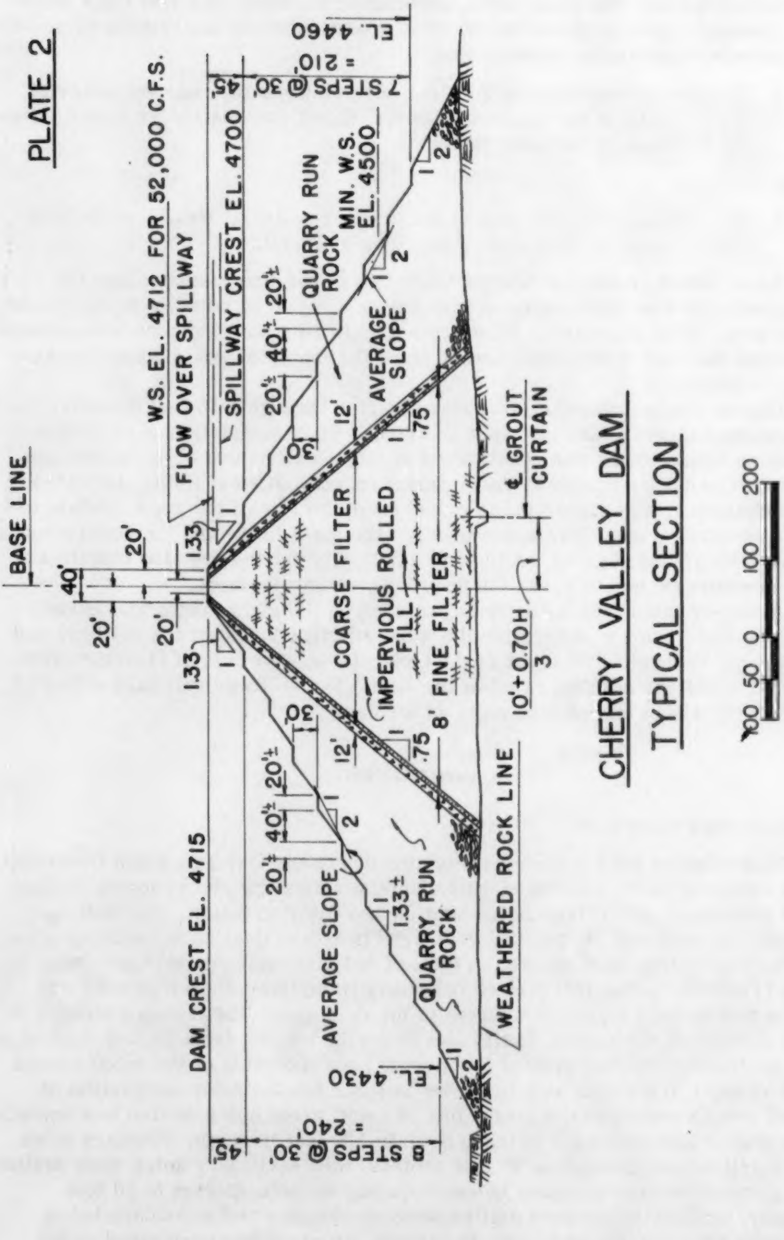
A side-channel type spillway is provided on the right abutment. Model studies were made to determine the most efficient layout of the spillway and discharge channel. The crest of the concrete ogee weir is at elevation 4700, 15 feet below the nominal crest of the dam. The spillway will pass a flow of 52,000 cfs with 3 feet of freeboard on the dam.

Construction

Foundation Preparation

In October of 1951 a contract was awarded to the Owl Truck and Construction Company in the amount of $684,480.00 for clearing and stripping of dam site foundation, cutoff trench, excavation and cutoff grouting. The entire foundation area was stripped to weathered bedrock, final clean up being performed by hydraulic monitors. A total of 291,500 cubic yards of overburden was removed. A cutoff trench to relatively firm, unweathered granite was excavated in the foundation adjacent to the dam axis. The trench averaged 20 feet in width at the bottom and 11 feet in depth, varying from 20 feet deep at the abutments to 4 feet deep at the stream bed. Location of the cutoff trench with respect to the dam axis is shown on Plate No. 2. After completion of cutoff trench excavation, a single line of cutoff grout holes on five foot centers was drilled and grouting by stages down to 120 feet in depth. Primary holes were drilled and grouted on 20 foot centers, then secondary holes were drilled and grouted between primary holes, reducing the hole spacing to 10 feet. Finally, tertiary holes were drilled between primary and secondary holes, reducing hole spacing to 5 feet. In general, grouting was performed on all holes so that stages in adjacent holes differed by no more than one stage. In

PLATE 2



CHERRY VALLEY DAM

TYPICAL SECTION

all cases grout used was neat cement grout with water-cement ratios ranging between 0.6 to 1 and 10 to 1. Grout pressures used averaged one pound per square inch per foot depth of hole. Grout takes in the various stages are shown in following table:

TABLE I

Unite Grout Requirement
Cu. ft. per Lin. ft.

Hole \ Stage	First 0 to 23'	Second 23' to 43'	Third 43' to 63'	Fourth 63 to 123'
Primary	2.366	0.745	0.097	0.056
Secondary	1.068	0.332	0.000	–
Tertiary	0.590	–	–	–

Additional foundation preparation under this contract consisted of backfill of prominent seams or crevices with a lean concrete under the compacted earth core to facilitate fill placement.

Dam Embankment

In October 1953, a contract for construction of the dam embankment in the amount of $7,162,800 was awarded to the Guy F. Atkinson Company. As soon as the necessary equipment could be assembled, a small quarry was developed just upstream from the damsite and construction of an upstream rock cofferdam commenced. This cofferdam was designed and constructed to be later incorporated into the upstream toe of the rock embankment. Final closure and diversion of the Cherry River was accomplished in June 1954, with the placing of a 30 foot thick blanket of impervious material on the upstream face of the cofferdam. The first lift of this impervious material was placed to an elevation about 30 feet above streambed by end dumping and dozing into comparatively still water. The impervious blanket was completed to the crest of the cofferdam, 70 feet above streambed, by placing in lifts about 2 feet thick. Compaction of the impervious blanket was limited to that obtained by hauling and spreading equipment. Seepage through the completed cofferdam was minor and was handled by a 1000 gpm electric pump.

Due to the slope of the streambed at and downstream from the damsite, as well as the low flow diverted (a maximum of 3200 cfs during construction in the first season), no downstream cofferdam was required.

Compacted Earth Fill

Under the terms of the contract, the Contractor was allowed the option of utilizing either one or both of two designated impervious borrow pits, either of which contained sufficient material for the embankment, as determined by auger borings. Both borrow pits offered a downhill haul to the damsite, and the Contractor chose to haul all of the fill material from a single pit, distant about three miles from the damsite, only slightly less than the distance to the alternate pit. The selection of pits provided a saving in haul road construction,

inasmuch as the haul road constructed passed just below the major rock quarry, one-half mile from the Dam, and was used also for trucks from the quarry. This arrangement proved satisfactory, although it was necessary to provide traffic control at the intersection of the quarry road and the main haul road much of the time.

Impervious material in the borrow pit selected by the Contractor was a decomposed granite derived by deep chemical weathering of inplace granitic bedrock. The area was known to contain several springs and was traversed by two small streams which flowed throughout the year. Moisture content of material in place ranged from 5% to 15% over the optimum required for compaction and this single fact to a major extent governed the Contractor's operations both in the borrow pit and on the fill.

Mechanically the borrow pit classification ranged from sandy silt to silty sand with 95% of the material classified as silty sand. Within the above ranges the material was further classified as follows:

Type	Characteristics	% finer than #200 screen	% of total Mtl Used
1.	Finely graded-Low Density	41 to 82	15%
2.	Fine graded-Above average density	34 to 41	35%
3.	Coarsely graded-Below average density	22 to 34	35%
4.	Coarse graded-High Density	10 to 22	15%

The average plasticity index of material in the borrow pit was 2.3 and ranged from 5.0 to 1.0. Of the samples tested 15% of the material had a P.I. of 5.0, 15% a P.I. of 3.0, 35% a P.I. of 2.0 and 35% a P.I. of 1.0.

Laboratory dry densities of the material ranged from a minimum of 113.3 lbs to a maximum of 128.0 lbs. per cubic foot when compacted at modified A.A.S.H.O. effort (56,250 ft. lb. per cu. ft.)

During the winter of 1953-54 the borrow pit, 200 acres in area, was cleared of trees and brush and stripped of humus and organic soil to an average depth of 2 feet. Haul roads were constructed, and some surface drainage excavation was performed.

Under the bidding schedule, the price established by the City for borrow pit excavation was $0.20 per cu. yd. This price included all excavation, whether waste or fill, and covered the cost of treatment to produce material of optimum moisture content, handling of stones and boulders in the pit, loading and hauling. The Contractors' bid price for placing and compacting material in the embankment to the specified density was $0.40 per cubic yard. Compaction results were specified; methods were left up to the Contractor.

In addition to the foundation treatment performed under the contract for stripping the damsite, under the fill contract, all joints and seams under the impervious section of the dam were slush grouted at least 48 hours in advance of placing fill thereon. Thus, foundation slush grouting proceeded up the dam abutments generally two days in advance of fill placement. Slush grout was a sanded grout composed of 1 part cement, 3 parts sand and 2 parts water.

From streambed to the crest of the dam, all impervious material placed

against the foundation was hand compacted for a distance of from 3 to 4 feet out from the foundation contact, using pneumatic tampers. The Contractor's bid price for this work was $6.00 per hour for each tamper used, maintained and operated, including sufficient hand labor to place material at the contact. A total of 6,430 tamper hours were used during the term of the contract. The area of the contact between the foundation and impervious core was 31 acres. During fill placement, the fill contact zone was kept to an elevation averaging three feet above the balance of the fill.

Impervious material was loaded in the borrow pit, transported to and placed on the embankment by 20 c.y. carryalls operating on the three mile haul road at speeds up to 40 mph. At the maximum rate of placement, about 24,000 c.y. per day, 20 of these units were used on each of two shifts. Carryalls were generally loaded in the borrow pit by two pusher tractors operating in tandem.

Continuous treatment of impervious material in the borrow pit was required to remove residual boulders encountered and to reduce moisture content to optimum for compaction. Material encountered in the pit varied in moisture content from 20% to 30%, and optimum moisture for compaction to specified density varied from 10% to 14%, depending upon gradation and specific density. Reduction of moisture content was accomplished by a systematic cultivation of the borrow pit and rotation on a 4 to 5 day cycle of excavation in designated areas of the pit. The cycle of excavation and treatment was approximately as follows:

> In a designated area where moisture had been reduced to optimum or slightly above, material was excavated to a depth of approximately 18 inches. On the day following, the area was ripped to a depth of about 2 feet by a crawler tractor equipped with a back-mounted ripper, and any boulders encountered were removed to the boundary of the area. On the next day the area was plowed with a two foot bottom plow. This treatment was repeated on the following day in areas of excessively high moisture content. On the next day the area was harrowed to a depth of about one foot by means of a spring tooth harrow. The following day the area was checked for moisture and if found satisfactory, excavated for fill in succeeding shifts.

All treatment for reduction of moisture content was performed on the day shift and the dryer areas of the horrow pit were generally reserved for hauling to the fill after sunset on the swing shift. Varying with the season and time of day, it was determined by testing that moisture content could be reduced by a maximum of 2% during the cycle of rock removal, leveling and sheepsfoot compaction on the fill embankment. Considerable experimentation with compaction units of various sizes and types on lifts of varying thickness was performed in early phases of impervious fill construction. During the early part of the first construction season for impervious fill, Summer, 1954, two rubber-tired compactors were tried, with gross weights of 50 and 100 tons, as well as sheepsfoot compactors ballasted to gross weights varying from 25 to 35 tons. Primarily because of the higher reduction of moisture content due to evaporation resulting from sheepsfoot compaction and secondarily because of the greater apparent compactive efficiency derived from increased weight, the contractor shortly standardized on the 35 ton sheepsfoot roller for all compaction.

A total of 1260 field density tests were taken during construction, averaging one for each 2,300 c.y. of impervious material placed in the embankment. The

average field density of all samples taken was 91% of laboratory density attained by the use of apparatus and procedures required for A.A.S.H.O. Method T 99-49, with the following changes:

a) the weight of the compaction hammer was 10 pounds instead of 5.5 pounds.

b) the hammer drop was 18 inches instead of 12 inches

c) the control sample was compacted in 5 equal layers instead of 3, using 25 blows per layer.

To obtain 91% average density the contractor settled on a compaction effort of 12 passes of a 35 ton sheepsfoot roller on each layer of 9 inches, loose depth.

The contractor placed impervious fill in the following sequence:

1. After compaction of a layer, the top surface was bladed off by means of a patrol grader. This operation was found to be essential in obtaining uniform thickness in the subsequent layer of material.

2. Just prior to placement of a layer, the top of the previous layer was scarified to a depth of about two inches to break up hard surfaces caused by the tires of hauling units and to insure bonding of the two layers.

3. A layer of material was placed on the fill by carryalls and spread to a thickness averaging 9 inches, loose measurement, by a crawler tractor equipped with a bulldozer.

4. The loose layer was raked through by a crawler tractor equipped with a rock rake to remove rock of a size over four inches. This operation was supplemented by a continuous patrol of a dump truck and two laborers following behind the rock rake to pick up oversize material which slipped through the rake.

5. The layer was compacted as noted above.

Although the results obtained by the above procedure were specified, the contractor was allowed a great deal of latitude as to methods, and followed the procedure outlined after considerable experimentation.

Transition Zone

The impervious core was flanked up and downstream by a transition zone 20 feet thick grading from sand and fine gravels adjacent to the impervious core to coarse gravel and cobbles. The required gradation for transition material was obtained by selective excavation from extensive deposits of sands and gravels in the flood plain of Cherry Valley. No mechanical grading of the material was necessary.

Transition material was placed in layers one foot thick, each layer being compacted by one complete coverage of a crawler tractor.

During the summer of 1954, placement of impervious and transition fill material progressed to an elevation considerably above the average elevation of the outside rock envelopes.

To retain the transition material on slopes of 0.75:1 and 0.7:1, it was necessary to place rock adjacent to the transition zones as the elevation of

the impervious and transition zones increased above that of the full rock section. Rock for this purpose was placed by dumping from the transition zone along the full length of the dam. This "retaining" rock was dumped without sluicing at the point of the dump to prevent damage to transition and core material. Rock dumped dry adjacent to the transition zone reposed on a slope approaching 1:1. In general, rock selected for dumping adjacent to transition zones was the smallest available from the quarries, and thus furnished additional transition in grading from coarse gravel and cobbles into quarry-run rock.

Rock Embankment

Rock for embankment was granodiorite quarried from three sites, all within a half of a mile from the damsite. Overburden to an average depth of 6 feet was stripped from quarry sites and wasted.

Chemical weathering of the rock extended to an average depth of 35 feet below bedrock surface, the extent and severity of weathering decreasing with depth. During construction of the embankment, some of the more severely weathered bedrock was wasted, although much of it was suitable for rockfill placed adjacent to the transition zones.

For all quarry excavation, including stripping and waste, the Contractor was paid $0.50 per cubic yard, the price being stipulated in the contract by the City, based on cross section measurement of the quarry sites. In addition, for all rock placed in the embankment, the Contractor was paid his bid price of $0.72 per cubic yard, based on embankment cross sections.

The Contractor experimented with various drill hole patterns and loadings and standarized on the following:

1. Drilled 4 inch diameter vertical drill holes, 30 feet deep, on an average spacing of 10 feet by 12 feet. Drills used were wheel-mounted, percussion type, wagon drills.

2. The bottom 4 feet of each drill hole was loaded with 60% gelatin, the balance of the hole with 40% gelatin to within five feet of the collar.

3. Explosives were detonated by means of electric blasting caps, delayed progressively from the working face back into the quarry.

The powder factor used averaged 0.6 pound per cubic yard of rock removed from all quarries.

Blasted rock was excavated from the quarry by means of 5-1/2 cubic yard electric, 3-1/2 and 2-1/2 cubic yard diesel shovels, and very little secondary shooting was required. Fifteen cubic yard, off-highway, rear dump trucks were used to transport blasted rock from quarries to the dam embankment.

In general, the rock fill portion of the dam embankment was constructed by end dumping on 30 foot vertical lifts, although in a few instances 15 foot lifts were used where access became a problem. To obtain the average design rock slope of 2 horizontal to 1 vertical and still save rehandling the surface rock during construction, the up and downstream rock faces were constructed on 20-foot wide horizontal berms at 30-foot vertical intervals, with the rock between berms taking its average material slope of repose of 1.33 horizontal to one vertical (See Plate 2).

Compaction of rock embankment was obtained by sluicing, using hydraulically operated monitors equipped with nozzles of 3, 3-1/2 or 4 inches diameter, operated at nozzle pressures ranging from 70 to 140 pounds per square inch. The skidmounted monitors were always placed out ahead of the rock dump on top of the previously placed lift 30 feet below, and were situated so that the water stream was directed into the face of the dump.

In dumping, rock was deposited on the edge of the dump and bulldozed over the edge, sufficient fine material being left on the top of each lift to provide an adequate surface for hauling. Prior to placing the next lift, this comparatively smooth surface was ripped up and the fine materials washed down into the embankment. Thus, in addition to sluicing the dump face, primarily at the point of the dump, each monitor was required to sluice the horizontal surface on which the rock was dumped.

Rock embankment was constructed on a two or three shift basis throughout the year, inasmuch as freezing conditions in the winter never became sufficiently severe to halt the work. The maximum rate of rock fill placement attained was about 14,000 cubic yards per day, using ten 15 cubic yard trucks on each of three shifts. Average rate of placement for the construction period was about 7200 cubic yards per working day.

The Dam embankment was completed in October, 1955; total time of construction was two years.

Performance

Reservoir Operation

Since the operation of the reservoir may influence the performance of the embankment, a brief description of the operation to date is given below.

The dam embankment was completed in October 1955. However, the spillway concrete was not completed until the fall of 1956, so that during the 1956 run-off season the water surface in the reservoir was kept below elevation 4650. This elevation is 50 feet below the spillway lip and 65 feet below the nominal crest of the dam. This was possible because the outlet valves have a capacity in excess of normal requirements in order to permit the reservoir to be operated for flood control. This excess capacity allowed the City to maintain the water surface at or below the permissible elevation during the run-off season. Of course, had a flood threatened during this period, it would have been necessary to drown out some of the Contractor's operations temporarily.

In 1957 the reservoir was completely filled. This also allowed a check on the operation of the spillway. Since the maximum inflow at the tailend of the run-off was only around 500 cfs the flow over the spillway was limited to this amount. The maximum elevation of the water surface was 4700.6, 14.4 feet below the dam crest. Under these conditions the spillway functioned without trouble, although the flows and heads really did not offer a very severe test.

In both of these years, 1956 and 1957, it was necessary to draw the reservoir down to elevation 4600 to allow other construction work in connection with the Cherry Power Project to proceed. As soon as the downstream reservoir of the Modesto and Turlock Irrigation Districts could receive the water, the outlet valves were opened. In 1956 the water surface was lowered from elevation 4650 to 4595, a total of 55 feet, in 15 days. In 1957 it was lowered 100 feet in 45 days. These conditions approach the classical case of

"sudden drawdown." However, since the water level was kept at its maximum elevation for only a short period of time, the core was not completely saturated, and the drawdown condition was not as severe as it would have been had the core been saturated.

It is expected that the reservoir will fill again this year, and as in the past two years it will again be drawn down to elevation 4600.

Measurements

Before filling of the reservoir was started, survey monuments were set in the impervious core along the crest of the dam. These monuments consist of a standard bronze survey marker set in an 8 inch diameter concrete base, extending 3 feet into the core material. These monuments were set at 400 foot intervals in a straight line along the crest of the dam. Reference points were set on the extension of this line at each abutment. The elevation of each point was determined and recorded. Bench marks were set at each abutment for future reference. The horizontal distances between the reference points on the abutments and the monuments along the dam crest were also measured and recorded. Settlement points were not established on the rockfill, neither on the crest of the dam nor on the berms.

Measurements have been made on these markers at irregular intervals since the reservoir was placed in operation. Readings have been taken rather frequently during run-off period when the reservoir fills quite rapidly and also during the period when it is being drawn at a greater than normal rate. At other times the interval between readings is much longer since conditions are pretty much constant and the embankment movements occur at a much slower rate.

The amount of settlement at each marker is measured. Also the deflection of the marker from a straight line along the theoretical axis of the dam. The horizontal distances from marker to marker have been checked several times to see if there is any apparent movement along the fill from the abutments toward the section of maximum height. To date there does not appear to be any motion in this direction.

A weir was also set in the old stream channel below the dam to measure seepage. This has not been too successful as described in more detail later.

Seepage

Seepage is probably the most direct indication of the success of this type of dam. It has been said that a stable dam embankment can be constructed out of almost any competent material provided that the resultant seepage is tolerable. What the tolerable seepage may be is something to be decided for each dam and depends on the basic function of the structure.

The design studies for Cherry Valley Dam indicated that maximum seepage through the impervious core might be of the order of 3 to 5 cfs. This is based on the water surface remaining at maximum elevation long enough to saturate the core and establish the phreatic line at its highest position. Since the coefficient of permeability of the core material is about 0.01 feet per day it would take a long time to achieve this condition. The State Fish and Game Commission requires that a minimum streamflow of 5 cfs be maintained downstream from the dam during the fall of year. Obviously it would be very nice if this flow could be maintained by seepage through the dam without the need of a small capacity auxiliary outlet. At any rate, for

Cherry Valley Dam it was decided that the maximum expected seepage would be tolerable.

Upon completion of the embankment, an attempt was made to measure the seepage by means of a weir downstream from the dam but upstream from the channel leading from the outlet valves. Unfortunately this part of the river channel has five to ten feet of semi-pervious material over the bedrock. This material consists mostly of fines washed out of the rockfill by the sluicing during construction. At the weir location the surface flow is concentrated in a well defined channel and it was hoped that the underflow could be cut off without an expensive structure. This was not the case.

That there is underflow is shown by the fact that for a constant reservoir elevation, the amount of flow as measured at the weir varies with the discharge of the outlet valves. As their discharge is increased, the increased depth of flow in the river downstream of the weir raises the water surface in the semi-pervious material and causes increased flow over the weir. When the outlet valves are closed, there is no flow over the weir. At such times none of the seepage reaches the surface of the semi-pervious material.

However, when the discharge of the valves was constant, the flow over the weir varied with the water surface elevation in the reservoir. A record has been kept of the flow over the weir since an increase in this flow with the other variables remaining constant would indicate an increase in seepage through or under the embankment.

Complicating the seepage measurements are the presence of several springs under the downstream portion of the rockfill. These springs were noted during construction. Dye tests were made to find out if there was a connection between them and Cottonwood Creek, which enters the reservoir just above the dam. No connection was found. Since the rockfill is free draining no remedial action was taken. These springs normally only flow during the spring and early summer. Their location has been recorded but it is not possible to separate their flow from the seepage measured at the weir.

There is also a stream gaging station about 3/4 of a mile below the dam. Measurements at this station include the flow of two creeks and several springs between the dam and the station, so that the readings are of little value in determining seepage of the dam.

To date, the seepage has been very small, probably less than 1 cfs. At times when the main outlet valves have been closed it has been necessary to open the small auxiliary outlet in order to maintain the required 5 cfs minimum streamflow.

Settlement

The settlement of a dam embankment is a matter of interest since it is usually considered to be an indication of the degree of compaction achieved during construction. In the case of rockfill, settlement may be taken as measure of the effectiveness of the sluicing in achieving rock-to-rock contact.

The vertical settlement of the points established on the crest of the dam is shown on Plates 3 to 8 herein. These settlement points are in the core material. They do not show the settlement of the rockfill. The rate of settlement of the core has paralleled the water surface elevations with a time lag of several months. As the reservoir fills, the rate of settlement increases. When the reservoir is drawn down the rate of settlement decreases.

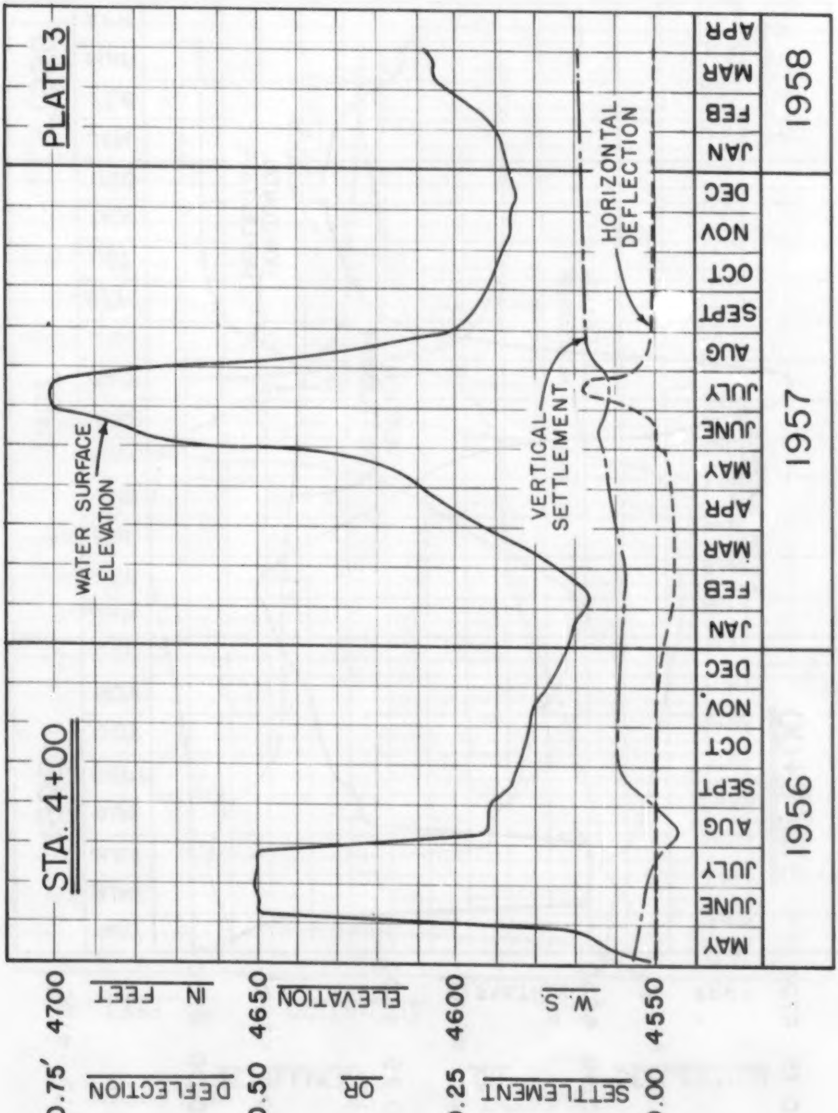


PLATE 3
STA. 4+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
0.75'
0.50
0.25
0.00
DEFLECTION OR SETTLEMENT
4700
4650
4600
4550
W.S. ELEVATION IN FEET
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV.
DEC
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
OCT
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DEC
JAN
FEB
MAR
APR
1956
1957
1958

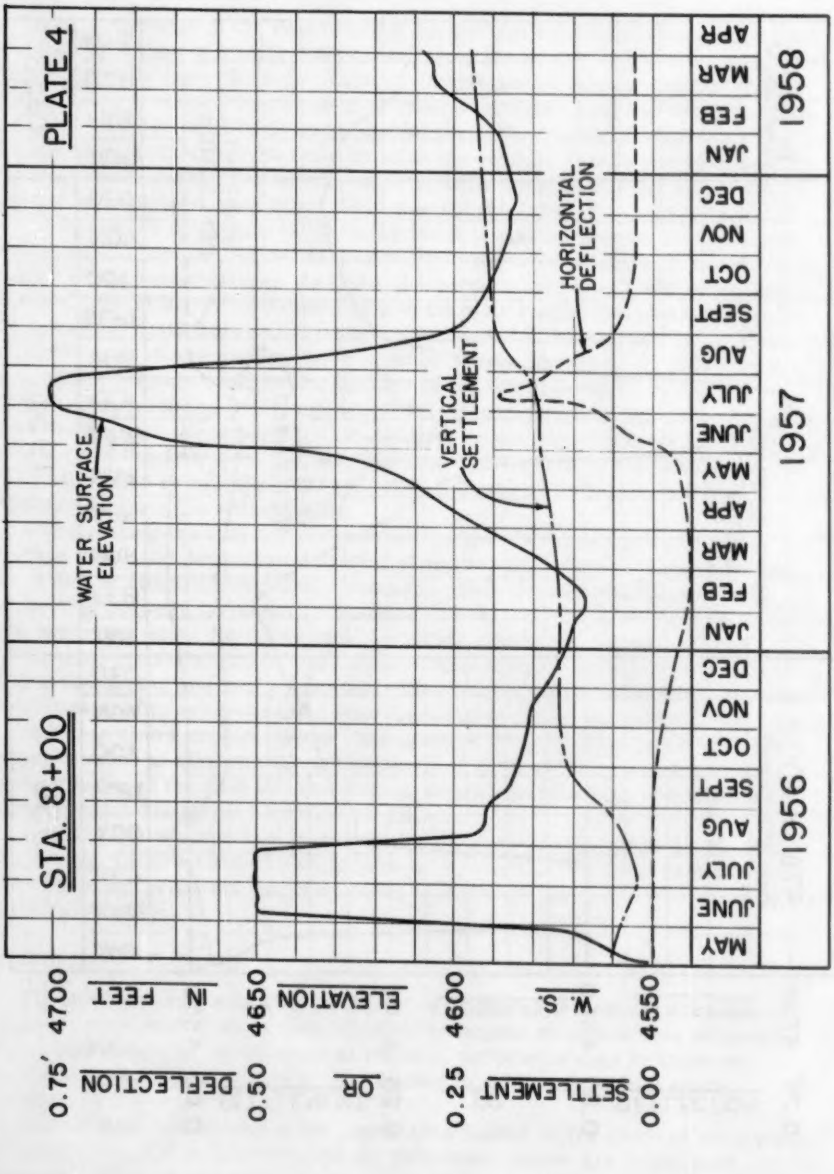


PLATE 4
STA. 8+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
SETTLEMENT OR DEFLECTION
ELEVATION IN FEET
W. S.
0.75' 4700
0.50 4650
0.25 4600
0.00 4550
MAY JUNE JULY AUG SEPT OCT NOV DEC
JAN FEB MAR APR MAY JUNE JULY AUG SEPT OCT NOV DEC
JAN FEB MAR APR
1956
1957
1958

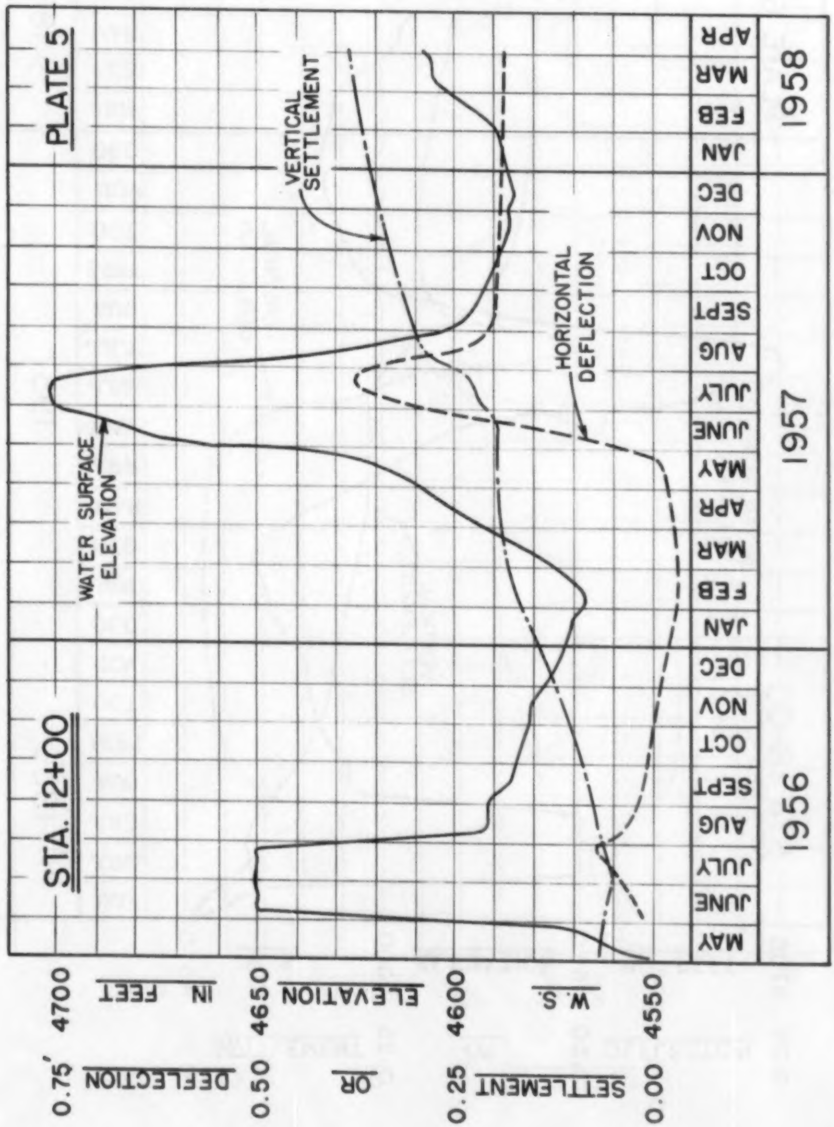


PLATE 5
STA. 12+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
1956
1957
1958
W. S. ELEVATION IN FEET
4550
4600
4650
4700
SETTLEMENT OR DEFLECTION
0.00
0.25
0.50
0.75'

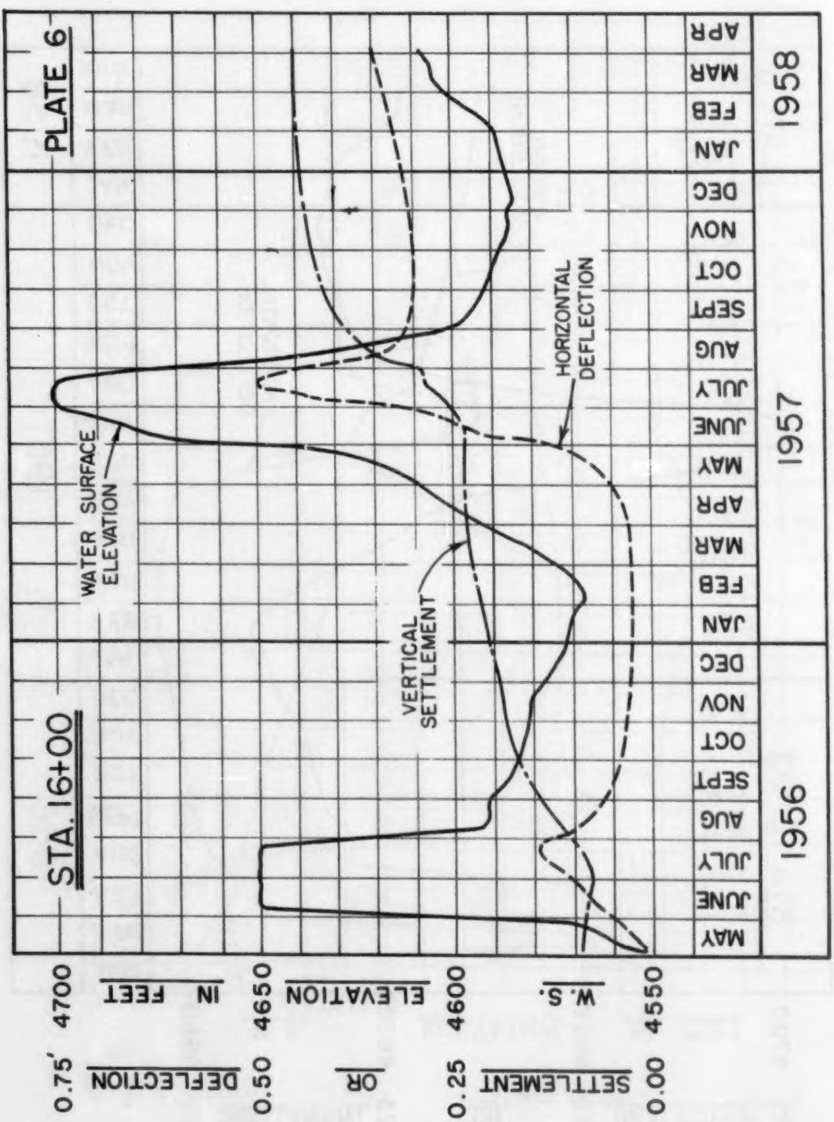


PLATE 6
STA. 16+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
SETTLEMENT OR DEFLECTION
0.00
0.25
0.50
0.75'
W.S. ELEVATION IN FEET
4550
4600
4650
4700
MAY JUNE JULY AUG SEPT OCT NOV DEC
JAN FEB MAR APR MAY JUNE JULY AUG SEPT OCT NOV DEC
JAN FEB MAR APR
1956
1957
1958

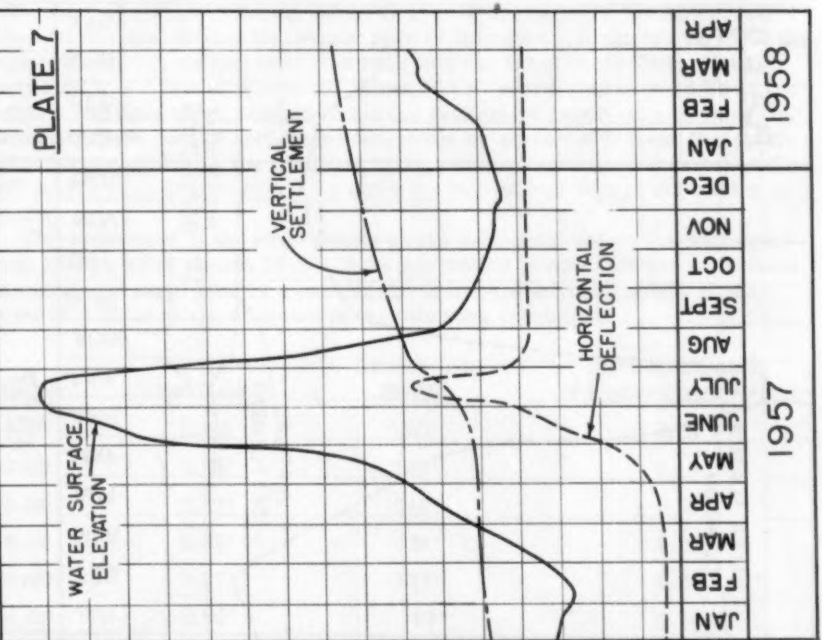


PLATE 7
STA. 20+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
MAY JUNE JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR
1956
1957
1958
4550 4600 4650 4700
W. S. ELEVATION IN FEET
0.00 0.25 0.50 0.75'
SETTLEMENT OR DEFLECTION

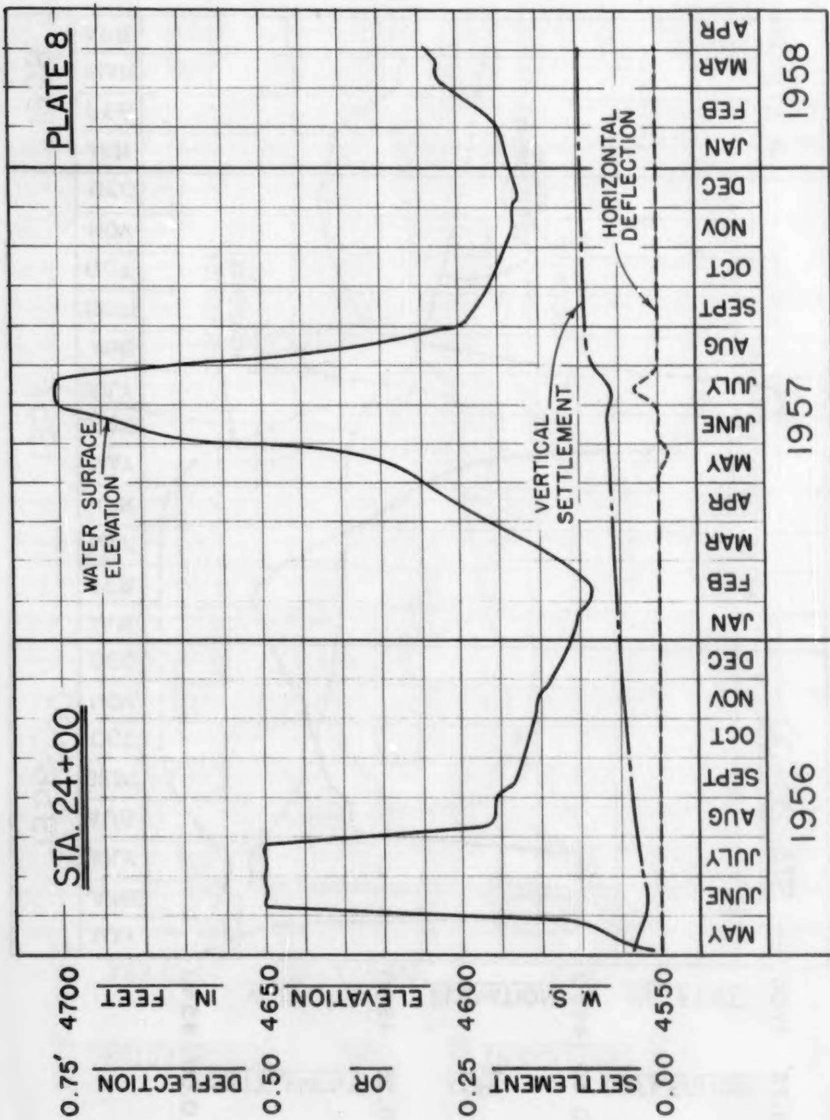


PLATE 8
STA. 24+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
DEFLECTION OR SETTLEMENT
0.75'
0.50
0.25
0.00
W. S. ELEVATION IN FEET
4700
4650
4600
4550
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
1956
1957
1958

The maximum total vertical settlement to date is 0.45 of a foot, a station 16+00 (the point of maximum embankment height). Of the total settlement, 0.23 of a foot occurred during the first year of operation of the reservoir, and 0.22 of a foot during the second year of operation. A camber of 1% of the vertical height of embankment was built into the crest of the dam. This amounted to 3.3 feet at this point. Thus, in two years operation 13.6% of the camber has been used. Judging from the settlement experience of other embankments, the rate of settlement decreases or ceases with time, the maximum occurring during the first few fillings of the reservoir. It is hoped this will be the case at Cherry Valley since a sway-backed dam is not a thing of beauty.

The settlement at the other points on the dam crest shows the same general pattern as at station 16+00. It is interesting to note that the settlement at each point expressed as a percentage of the embankment height at the point is quite uniform as shown in the following tabulation:

Station	Settlement	Embankment Height	Settlement as % of Embankment Height
4+00	0.10'	80'	0.12
8+00	0.23'	170'	0.13
12+00	0.37'	236'	0.16
16+00	0.45'	323'	0.14
20+00	0.37'	227'	0.16
24+00	0.09'	86'	0.10

As mentioned before, settlement points were not established on the rock fill. Visual observations indicate that the settlement of the rock fill has been greater than the settlement of the core. For example, the berms on the up and downstream faces of the dam were horizontal when built. They are now lower in the central portion of the embankment than at the abutments. The maximum settlement is about 2 feet, measured on the upstream rock berm at elevation 4580. This is probably because the maximum vertical height of rock embankment above either bedrock or the compacted core occurs at this berm.

At the dam crest there is additional evidence of differential settlement between the rock and the core. The crest was constructed to a 40 foot width. The central 20 feet of the crest is the core of the dam, the outer 10 feet on each side is composed of the 2 transition zones plus the rock envelope. The entire crest of the dam is covered with a 6-inch layer of fines and gravel, compacted and bladed to form a roadway for light traffic. After the first filling of the reservoir several longitudinal cracks were noticed along the line of the junction of the core and the transition zone.

These cracks were evidently caused by differential settlement between the rock (including the transition zones) and the core. As the rock settles it has a small component of movement in the upstream (or downstream, as the case maybe) direction. The cracks were not over 1/4 inch wide, and none of them extended continuously more than 30 or 40 feet. There were more of them on the upstream side of the crest than on the downstream. This would be expected since the water action on the upstream rock fill would cause some

additional settlement, despite the sluicing performed during construction. The cracks could not be traced more than 10 or 12 inches below the surface. Their locations were recorded and the roadway material was bladed to seal them and prevent entry of water into the fill.

After the second filling of the reservoir these longitudinal cracks were again noted. There was no marked connection between the location of these cracks and the ones in 1956. As before, a record was made of the locations of the cracks and they were filled by blading.

Another indication of this differential settlement is seen at the crest of the dam. The crest was originally constructed with a slight crown in the transverse direction. Due to the increased settlement of the rockfill this crown has increased somewhat, but an accurate measurement of this is not possible since the edge of the fill is quite irregular due to the size of the rocks.

Deflection

A certain amount of movement in a downstream direction is to be expected in any dam embankment as the reservoir is filled. What is not expected is that the embankment move upstream as the reservoir is emptied. Yet that is what has happened at Cherry Valley Dam during its first two fillings. This is depicted graphically on Plates 3 to 9, on which the dashed line represents the horizontal movement of the survey points (positive in the downstream direction).

Considering station 16+00 again, during the first filling of the reservoir the total movement downstream was 0.14 of a foot. When the reservoir was drawn down, this point moved back upstream again so that just prior to the second filling the residual downstream movement was 0.035 ft. This is a rebound of 0.105 ft. or 72% of the original movement. The rebound was quite rapid at first, closely paralleling the change in water surface.

During the second filling of the reservoir this point moved downstream 0.465 of a foot for a total movement (from empty reservoir) of 0.500 of a foot. (Remember that the first filling of the reservoir was only to elevation 4650, the second filling was to maximum pool at elevation 4700). When the reservoir was drawn down the rebound was 0.195 of a foot, for a residual deflection of 0.305 of a foot. This rebound amounted to 43% of the deflection occurring during the second filling, or 39% of the total deflection from empty reservoir.

The other points moved in a manner similar to that at 16+00 except that during the first drawdown of the reservoir they rebounded to a position upstream from their initial position with an empty reservoir. The amount of such movement at these points was:

Sta. 4+00	0.025 ft.
8+00	0.055 ft.
12+00	0.035 ft.
16+00	none
20+00	0.030 ft.
24+00	0.017 ft.

This phenomenon is hard to explain. The thought immediately occurs that an error was made in establishing the points. However the series of readings made on these points prior to, and during, the first filling of the reservoir show no indication that such was the case. The other idea that has been

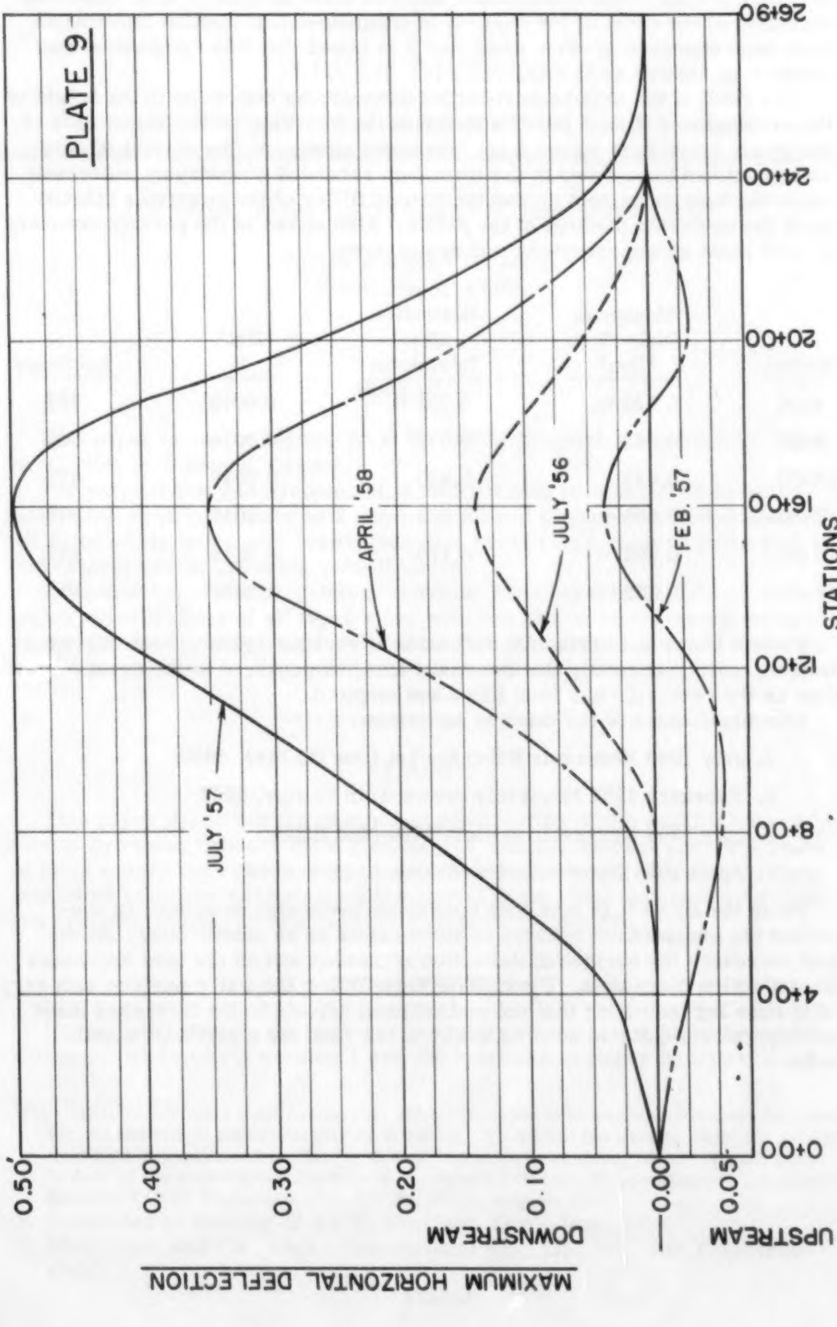


PLATE 9
MAXIMUM HORIZONTAL DEFLECTION
UPSTREAM
DOWNSTREAM
0.05
0.00
0.10
0.20
0.30
0.40
0.50'
STATIONS
0+00
4+00
8+00
12+00
16+00
20+00
24+00
26+90
JULY '57
APRIL '58
JULY '56
FEB. '57

advanced is that due to unequal settlement between the upstream and downstream portions of the embankment there is a net movement in an upstream direction at the crest of the dam. It is understood that similar movements have been observed in other dams and it is hoped that this symposium may produce an answer as to why.

The ratio of the maximum recorded downstream deflection to the height of the embankment at each point is shown in the following table. In the case of the points which have shown a net movement upstream, the amount of such movement has been added to the maximum recorded downstream movement since the total movement during the second filling of the reservoir started from the upstream position of the points. Also shown is the percent recovery at each point as the reservoir was drawn down.

Station	Maximum Deflection "Dm"	Deflection after Drawdown	"Dm" / H	Recovery
4+00	0.120 ft.	0.025 ft.	0.0015	79%
8+00	0.245	0.075	0.0014	70%
12+00	0.415	0.220	0.0018	47%
16+00	0.500	0.305	0.0016	39%
20+00	0.295	0.170	0.0013	42%
24+00	0.052	0.017	0.0006	67%

Plate 9 shows the horizontal deflection at various dates. These curves illustrate quite graphically the movement that has occurred at the crest of the dam as the reservoir has been filled and emptied.

The significance of the dates is as follows:

1. July 1956 Reservoir filled for 1st time (to elev. 4650)
2. February 1957 Reservoir drawn down to elev. 4567
3. July 1957 Reservoir at elev. 4700 (2nd filling)
4. April 1958 Reservoir drawn down to elev. 4608

From the above it is seen that insofar as horizontal movement is concerned the embankment behaves to some extent as an elastic body. As the load increases the horizontal deflection increases and as the load decreases the deflection decreases. These deflections follow the water surface with very little time lag indicating that the embankment adjusts to the increased loads quickly, but not quite as soon as would be the case for a perfectly elastic body.

Paper 1734 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM[a]

Torald Mundal,[1] M. ASCE
(Proc. Paper 1734)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

This paper describes the design and construction of the rockfill dam portion of Brownlee Hydroelectric Project. Of unusual interest are: diversion of flood waters over the partially completed embankment; inclusion of large quantities of compacted small rock material in the dam; and construction of the embankment on 110 feet of river-deposited materials.

INTRODUCTION

Three hydroelectric projects are currently scheduled by Idaho Power Company to ultimately develop 1,185,000 kilowatts of power by utilizing a

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1734 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at Meeting of ASCE, Portland, Ore., June, 1958.

1. Vice Pres. and Chf. Engr., International Eng. Co., Inc., San Francisco, Calif.

602-foot drop in the Hells Canyon reach of the Snake River along the Oregon-Idaho border. These projects, in downstream order, are Brownlee, Oxbow, and Hells Canyon. (Fig. 1) Brownlee and Oxbow projects, presently under construction, have completion dates of 1958 and 1960, respectively.

Brownlee Hydroelectric Project is situated in the upstream end of the 6,000-foot deep Snake River gorge, very appropriately named "Hells Canyon." The rugged terrain presented challenging design and construction problems, solutions of which resulted in several innovations in the field of rockfill dams. Project features other than the dam are discussed only because of some direct relationship with the dam.

Basin Description

The 72,000-square mile drainage basin upstream from the damsite is divided into three distinct areas. The upper area rises to a maximum elevation of 13,700 feet on the peak of Grand Teton. Precipitation falling on this area builds an annual snow pack which supplies a major portion of the summer flow of the Snake River.

The middle area contains many acres of irrigated farms as well as forests and brushland. This area, with its vast underground storage reservoirs, collects much of the annual rainfall for release through thousands of small springs.

The lower area, immediately upstream from the dam, is composed predominantly of barren, precipitous hills. Runoff from these hills, caused by the seasonal and often intense rainfall, produces many flash floods.

Water flowing past the damsite is not only regulated by the natural underground reservoirs, but also by numerous upstream reservoirs which have been constructed on the main stem and tributaries. As a result, the minimum flow of the Snake River at the Brownlee site is ideally suited for the generation of power. The minimum monthly average discharge recorded at the Oxbow gage was 7,000 cfs, while the mean discharge for the past 32 years was 17,000 cfs.

Site Description

The Brownlee damsite, about 70 miles from Weiser, Idaho, is located on the Snake River approximately 80 miles upstream from the confluence of the Snake and Salmon rivers. At the dam base line, the canyon is about 600 to 700 feet wide at river level and slopes up on both abutments at about 1 on 1; but the rock nose forming the left abutment just downstream from the base line has slopes as steep as 1/4 on 1. Rock outcrops and geological formations at the site indicate that bedrock lies a considerable distance beneath the valley floor.

Geologically, the project area is covered with thick beddings of miocene basaltic lava flows of the tertiary age. These beddings, as shown in Fig. 3, occur in massive layers which vary in thickness from 60 feet to more than 150 feet. The fine-grained basalt rock is dark grey when fresh, and becomes dark brown when exposed to air for any length of time. It contains crystals of feldspar in sizes from 1/4 inch to 1 inch in various beddings. By nature of its structure, the rock is hard, tough, and impermeable. Between the beds of lava flows, there are thin layers of tuff, varying in thickness from a few inches to a few feet.

At the damsite, the river cuts through five distinct beds which dip generally about 13° toward the west and form the easterly limb of a large syncline. The lowest of the beds is the most massive and extends across the river, underlying a major portion of the dam foundation.

The basalt rock layers contain natural joints which were formed during cooling of the lava flows and by movements of the crust of the earth. All beddings are closely jointed. The characteristic jointing and columnar formation was clearly evident on the left bank.

There were a few faults of small displacement in the area. These were considered to be inactive and therefore not a serious problem.

Of the few dikes at the damsite, the most noticeable was Dike No. 4, which cuts diagonally across the dam foundation and through the rock nose on the left abutment.

Project Description

Major features of the project are a rockfill dam, spillway, outlet works, and power facilities. (Fig. 2)

The dam has a crest length of 1,400 feet and a maximum height, as measured from bedrock to the top, of 400 feet. In section, the dam has a sloping impervious core flanked by sand and gravel filter zones and outer zones of rock. (Fig. 4)

The spillway, which is located in the rock nose of the left abutment, consists of an approach channel, a concrete control section, and a lined chute. Spillway discharges are regulated at the control section by four radial crest gates.

The outlet works consist of three gate-controlled openings through the control section of the spillway. Releases from the outlet works discharge into the spillway chute.

The power facilities include an intake channel, a power intake structure, individual steel penstocks in tunnels through the right abutment, and an outdoor type powerhouse. An initial installation of four units will be increased to six in the future, with an ultimate capacity of 540,000 kilowatts.

Foundation Explorations

The program of foundation explorations, conducted during preliminary and final stages of design, served to determine final design decisions. Of a total of 32 core-drill holes, the first eleven provided the data used in the selection of type of dam. Nine of the eleven were drilled in the river channel and two in the right abutment. There was a high percentage of recovery of all cores except those taken from tuff layers between the beddings and at breccia zones. Water tests of the holes indicated that only normal foundation grouting would be required.

The overburden was of considerable depth in the river channel with a maximum of 110 feet. It was generally well consolidated, and consisted of sand, gravel, cobbles, and small boulders. A deposit of sand and silt was found at the downstream end of a gravel bar near the left abutment.

Samples from some of the original holes indicated the presence of fine sand. If a rockfill dam were constructed at this site, seepage through the

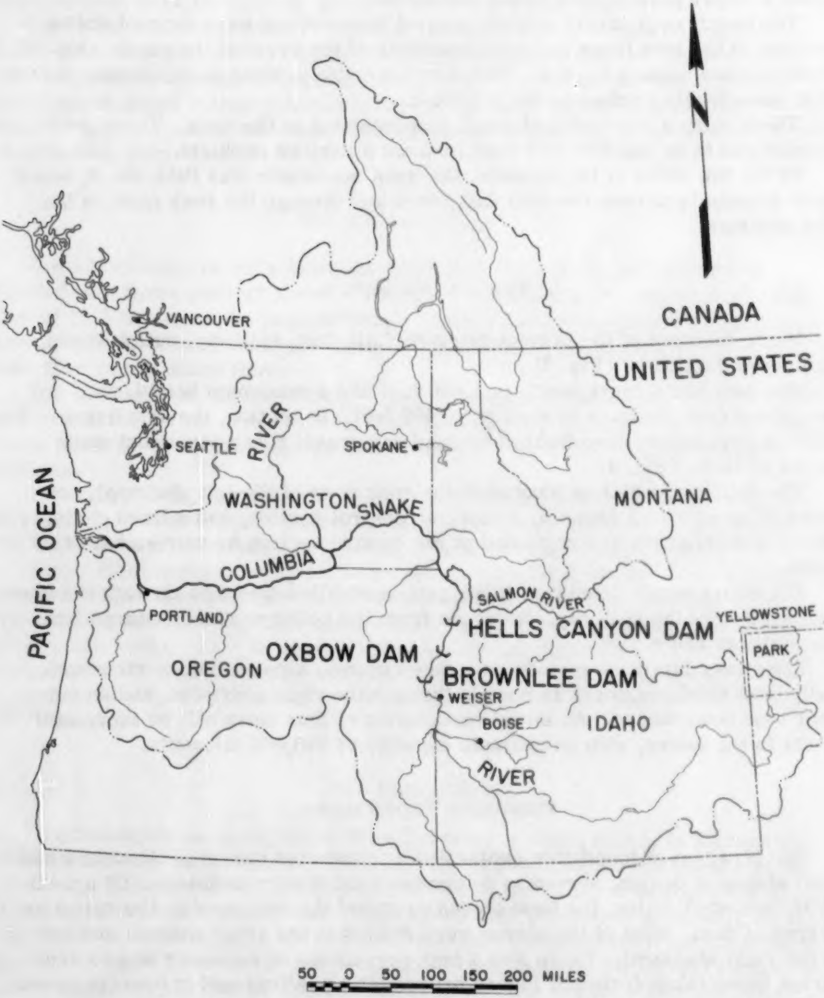


FIGURE I
BROWNLEE PROJECT
LOCATION MAP

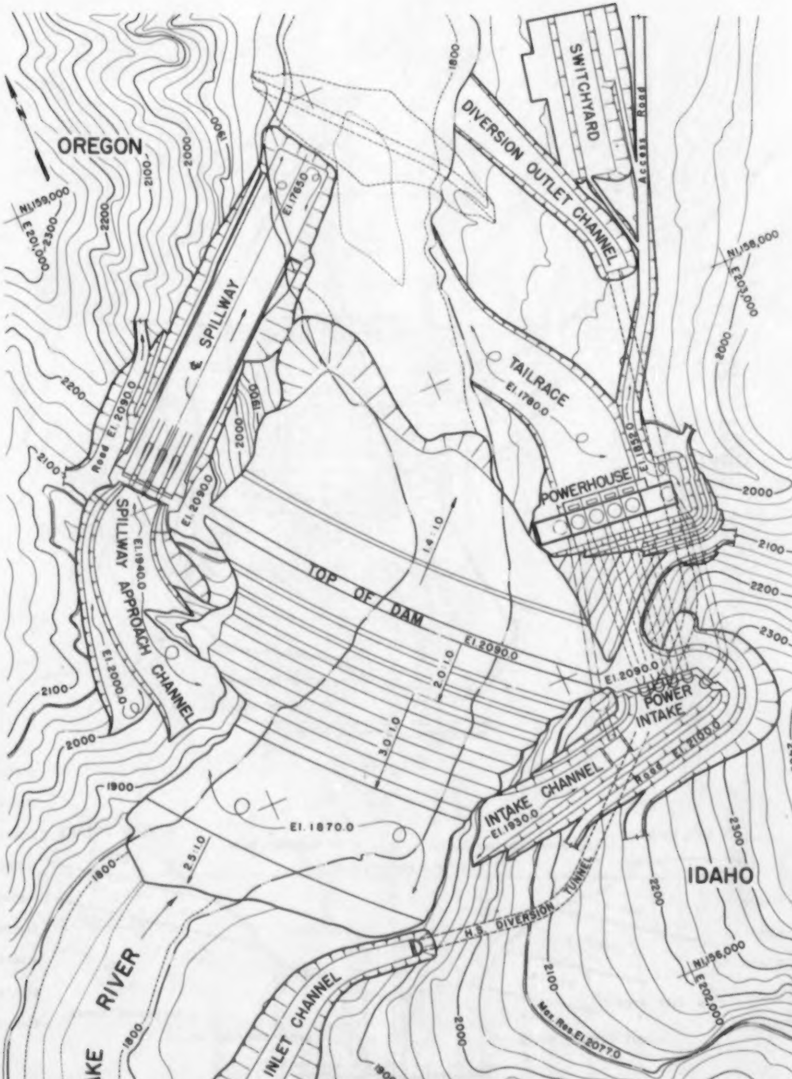
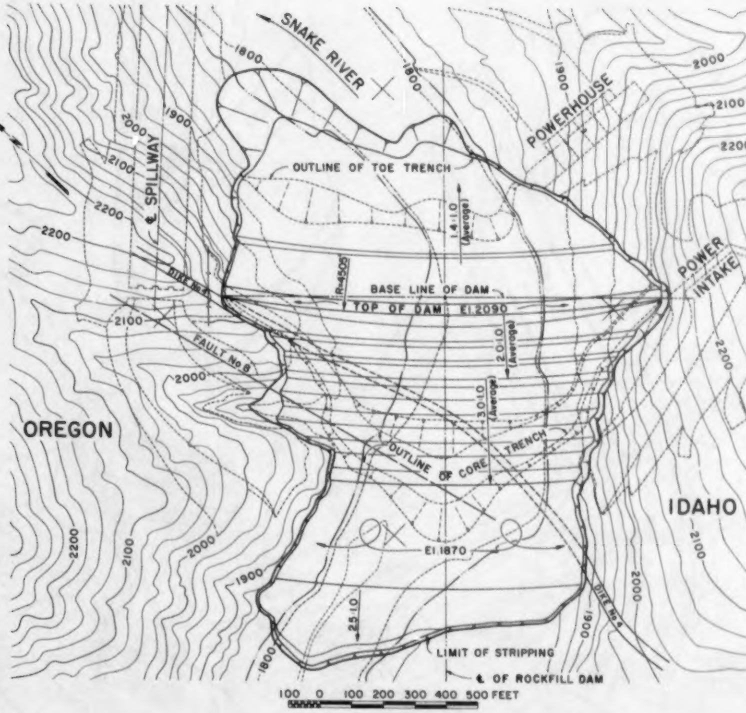
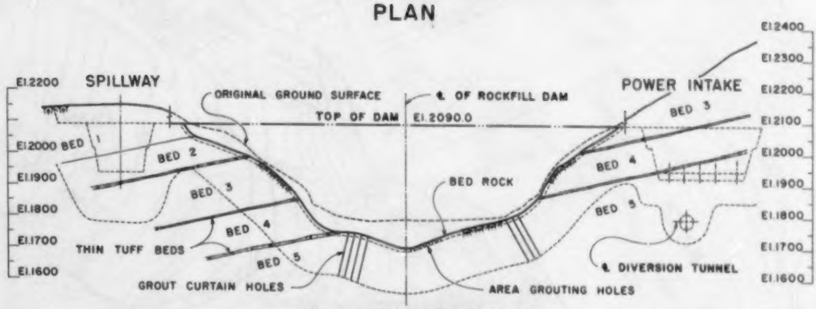


FIGURE 2
BROWNLEE PROJECT
GENERAL PLAN



PLAN



PROJECTED PROFILE ON ℄ CORE TRENCH

FIGURE 3
BROWNLEE PROJECT
ROCKFILL DAM

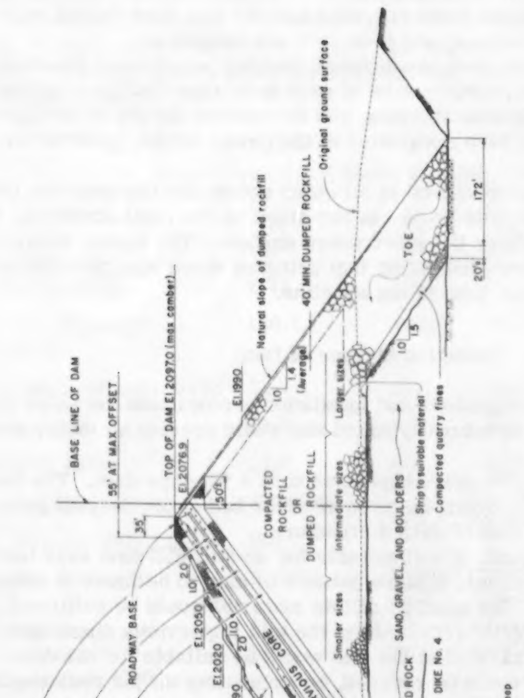


FIGURE 4
BROWNLEE PROJECT
ROCKFILL DAM-MAXIMUM SECTION

dam and foundation could cause piping in the sand or liquefaction of the fine sand upon being saturated and subjected to load. Therefore, additional holes were drilled near those preliminary holes which had indicated the existence of fine sand. The additional holes revealed that the fine sand layers were limited in extent and thickness, and therefore not dangerous.

Rock samples obtained from the different bedding zones were carefully studied to determine the probable size of rock to be expected from normal excavation practices. Special attention was directed to the dip of the beddings in arranging the unusual deep excavation at the power intake, powerhouse, and the spillway.

Artesian water was encountered in all holes except the two near the left abutment downstream of Dike No. 4. In two holes on the right abutment, water stood at 30 and 58 feet above the river water surface. The higher water level was farther from the river, indicating that artesian water was percolating down the dip from the east limb of the syncline.

Selection of Type of Dam

The preliminary investigations and foundation explorations revealed that the rock formation was structurally sound and would present no major problems.

A concrete dam would be more expensive than a fill-type dam. The hundred feet of concrete dam below the present river bed at the deepest point would nearly double the cost of such a structure.

There is no large deposit of soil suitable for an earthfill dam near the damsite. There are, however, shallow patches of clay to be found in small deposits in the vicinity. The quantity of this material would be sufficient for the small amount of material required for the thin impervious diaphragm in a rockfill dam. Rock excavated at the site would be suitable for the dam. In addition, project cost could be reduced, by excavating all the rock required in the embankment from other features of the project. Increasing the length of the channel to the power intake structure reduces the penstock lengths, thus simplifying regulation of the units without increasing the dam embankment cost. Based on the above considerations and further detail studies of the elements of cost, the rockfill dam was selected as the most economical type to build at this site.

Investigation of Construction Materials

Following selection of the type of dam to be constructed, a soils laboratory was established at the site and a construction materials investigation program was initiated. Before the field soils laboratory was completely equipped to perform all tests, the soils laboratory of Washington State College was retained to make independent tests and to check the field laboratory work.

All the basic material components of the dam, such as the impervious core, filters and rock were thoroughly investigated before start of construction.

Impervious Materials

Extensive explorations in the vicinity of the damsite eliminated all but two potential sources of satisfactory impervious materials. Development of one

of these sources would require construction of a 2-1/2-mile road on a six percent downgrade to the dam, while the other source would require processing of the material for removal of interspersed rock and gravel. Comparative cost studies indicated that development of the source requiring construction of the haul road would be the more economical.

Test results of typical samples from the selected site revealed the following characteristics:

Impervious Core Material Test Results

	B1 148	B1 149	B1 151	In Place Fill Material Text #1
Specific Gravity	2.78	2.90		2.85
Max. Dry Density* (lb/cu ft)	100.1	104.5	119.3	110.0
Optimum Moisture (%)	23.7	21.5	14.6	19.6
Liquid Limit	53.0	31.5	32.6	30.6
Plasticity Index	27.0	8.0	16.4	8.5
Permeability (cm/sec)	3.7×10^{-7}			2.1×10^{-5}
at Rel. Density (%)	90			90
Cohesion (lb/sq ft)	3,400	2,450		
ϕ^{o}	15	19		16.7
Mech. Analysis** (% passing #200 sieve)	90	39	54	64

*Modified AASHO

**Mechanical analysis curves are shown in Fig. 5.

As indicated by the test results, the impervious material has a low friction angle, but very high cohesive strength. In addition, the material has the property of being exceedingly plastic, a characteristic considered to be very desirable for a thin core material.

The natural moisture content of the material varies from 20 to 25%, slightly greater than optimum.

Filter Material

Satisfactory sources for fine filter materials in their natural state were not found in the Brownlee area. The best material available was in a large sand and gravel deposit located about three miles upstream from the site, on a bench approximately 100 feet above the river. Under a layer of top soil several feet thick, the material varied in composition from fine sand to large gravel. As may be seen from the mechanical analysis shown in Fig. 6, the material lacked sufficient amounts of fine and medium sands to meet the requirements for fine filter. Therefore, a processing plant was erected to

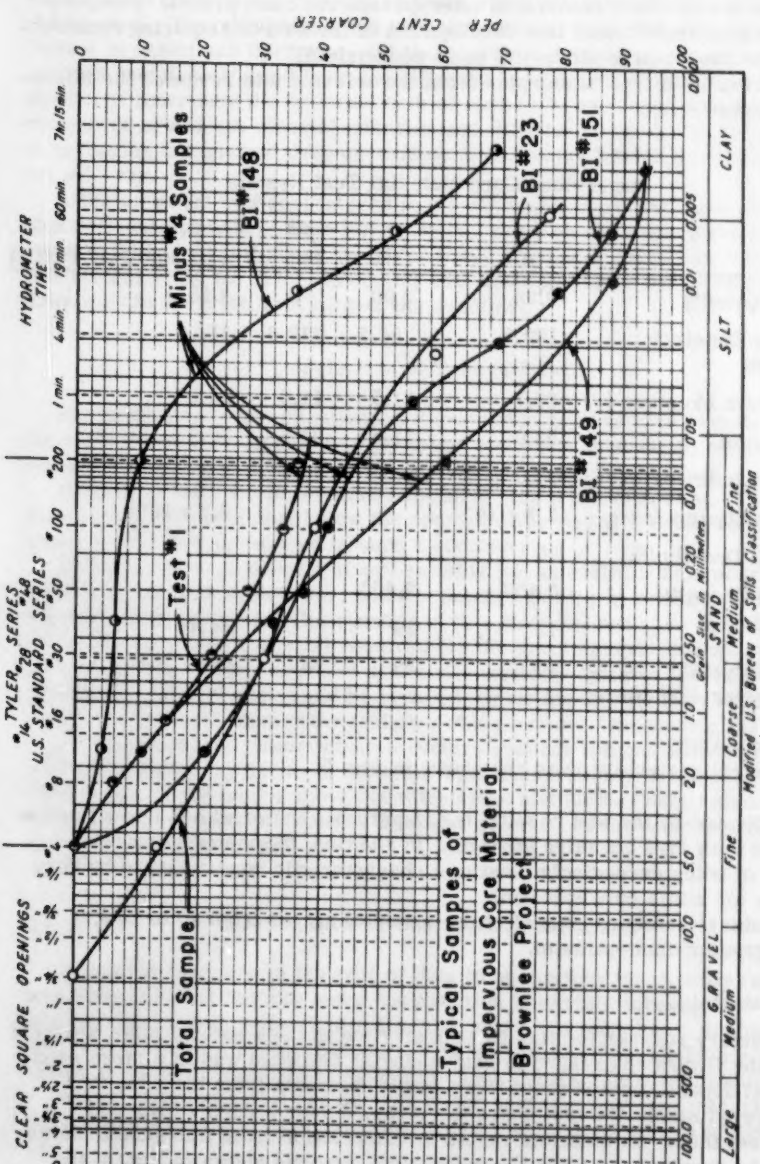


FIGURE 5

IMPERVIOUS CORE MATERIAL MECHANICAL ANALYSIS CURVES

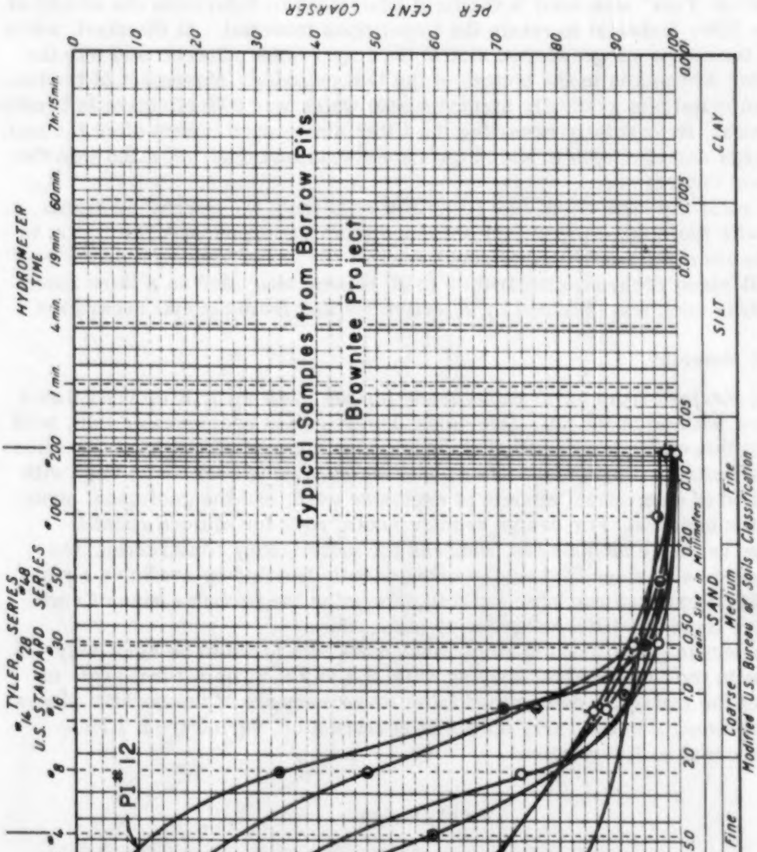


FIGURE 6

NATURAL FILTER MATERIAL MECHANICAL ANALYSIS CURVES

screen out coarse material larger than 1-1/2 inches, and to blend the remaining material with fine river sand. This resulted in a satisfactory grain-size distribution. Mechanical analysis of the resultant material is shown in Fig. 7.

A "Wash Test" was used in the field laboratory to determine the ability of the fine filter material to retain the impervious material. At the start, water rushed through a wedge-shaped slot in the impervious material and into the fine filter compacted in the bottom of the test cylinder. Movement of the impervious materials gradually formed a seal which was 90% effective in twenty-four hours. By carefully removing the filter material in layers after the test, the amount and the depth to which the impervious material migrated into the filter was determined.

The naturally-deposited, fine filter material from the borrow area was mixed with the large-size gravel screened from the processed fine filter, to form the coarse filter material required.

Small-sized rock, ranging from 3 to 10 inches, was used as a third filter zone. This rock was obtained by selective loading from normal excavation.

Rockfill Material

Rock obtained from excavations made for the main structures constituted the major portion of the fill. Geological investigation had revealed that, with the exception of the weathered surface rock, all the rock from the excavations would be suitable for dam construction. The excavated rock sizes vary with the method of excavation, amount of explosive used, and the geological structure of the bedrock. For design requirements, rock for sluiced rockfill should be as large as possible, whereas for satisfactory construction, the rock should be of sizes that can be economically handled by available equipment. Trial excavations were made to determine the probable size of rock obtainable from the different areas of excavation.

The specific gravity of the solid basalt rock was found to be relatively high, due to the iron content, varying from 2.8 to 2.9. The natural angle of repose of the material, determined from measurements of the rockfill slopes at both abutments in the early stages of construction, varied from 1.35 to 1.40 horizontal to 1.00 vertical.

Final Design

Final design of the dam was governed by the criteria as listed below:

1. Arrangement of the basic project layout for the most effective use of topographic and geologic conditions.
2. Sufficient design flexibility to permit modifications required by changes in foundation conditions or material properties which might occur as construction progressed.
3. Fullest possible utilization of readily available construction materials.
4. Complete analyses of the main embankment for stability and expected settlement.

Project Arrangement

Arrangement of the project features was governed primarily by the topographic prominence of the rock nose which forms the left abutment, and by

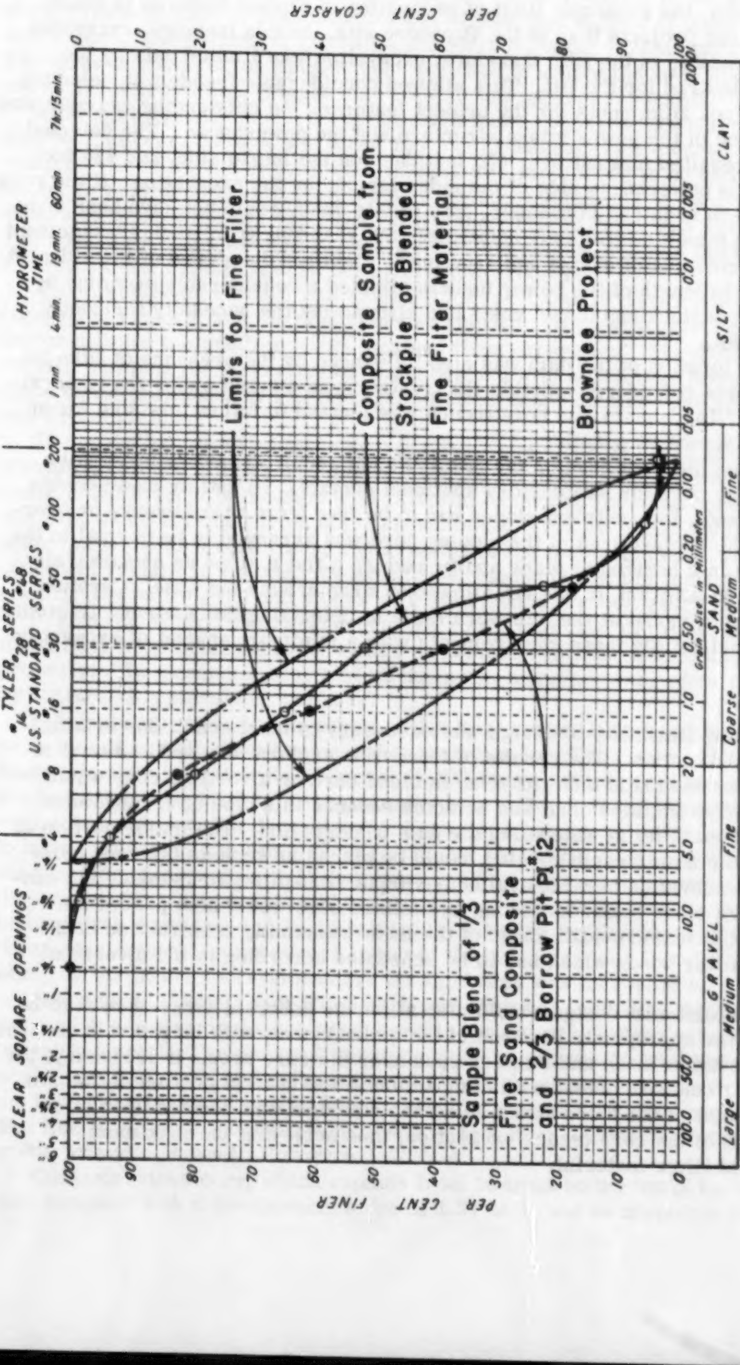


FIGURE 7

FINE FILTER MATERIAL MECHANICAL ANALYSIS CURVES

the soundness of the rock in the right abutment. The rock nose provided an ideal location for a spillway, while the massive rock forming the right abutment was well suited for boring of the power and diversion tunnels.

Normally, the economic limit of excavation of project features is much less at other projects than at the Brownlee site. In this instance, excavated rock was usable in the fill; therefore, excavation was limited only by the amount required for the fill. This allowed considerable freedom in selecting locations and alignments for the project features. An outstanding example was the location of the power intake structure and the powerhouse. The selected location resulted in a cut over 450 feet deep on the intake side, and 420 feet deep on the powerhouse side of the ridge formed by the excavation. If the rock had not been required in the dam, cuts of this depth would not have been economically feasible. The only additional cost resulting from such arrangement was the provision of drains and grouting to stabilize the slopes of the ridge. A reduction in length of the power tunnels effected a considerable economy by reducing penstock and tunnel costs and eliminating the necessity for costly surge tanks.

Actual location of the dam was also influenced by the presence of Dike No. 4, which was considered suitable for treatment as a cutoff under the impervious core. The impervious core trench was located to follow this dike for as great a distance as possible.

Utilizing the rock nose on the left abutment required particular care in aligning the core for satisfactory abutment contact. To satisfy basic design requirements, the abutment slopes had to be free from sharp breaks or overhangs. The contact area of the sloping core was arranged to be normal to the core zone in a horizontal plane and to have as a flat a slope as economically feasible. In addition, it was not allowed to wrap around the nose. Careful studies and a model at the site were made to determine the abutment trimming which would satisfy these requirements with a minimum amount of excavation.

Flexibility

Providing flexibility of design was necessary to meet conditions arising during construction. The results of foundation explorations and material investigations indicated that either an inclined core or a vertical core would be suitable. The inclined core was selected because this type of arrangement provided flexibility in scheduling the dam construction. The major portion of the dam, the downstream rockfill, could be placed as soon as the rock from required excavation became available without stockpiling or delaying the construction of other features. Using an inclined core would also permit the placement of main rockfill ahead of the time-consuming construction of the core and filter zones which had to be scheduled according to the diversion program.

The embankment design should also allow the different sizes of rock to be placed at the appropriate portions of the embankment, with larger rock dumped near the outside faces and smaller rock near the core area. If necessary, the small rock could be placed with compaction in the central portion of the dam.

The extent of the filter zones below the riverbed was decided upon after completion of the core trench excavation, thus achieving the maximum economy of filter material.

Utilizing Available Material

Almost any excavated rock from the thick basalt beds at the site would have resulted in satisfactory material for the rockfill portion of the embankment. The selection of zone thicknesses, slopes, and method of placement was based on the estimated gradation and yield from the various excavations. The three portions of the embankment designed to utilize the rock material were: downstream toe, downstream rockfill and upstream rockfill.

Although hydraulic model tests indicated that no apparent downstream erosion would occur, the fact that about 100 feet of natural river-deposited material underlies the rockfill embankment was considered to be sufficient cause, to provide special toe protection. To accomplish this, a toe trench, excavated to bedrock and backfilled with selected large rock, was designed. The rock zone in the toe was to have a minimum section 172 feet thick, with a 20-foot filter blanket against the riverbed overburden.

The downstream rockfill was designed for two types of construction: one compacted rockfill, and the other dumped rockfill. The compacted rockfill design utilizes the smaller rocks obtained from excavation. The dumped rockfill utilizing the larger rocks, was designed with an average downstream slope of 1.4 horizontal to 1 vertical, the flattest slope obtained during field test investigations.

The upstream rockfill has an average upstream slope of 2 on 1 above Elevation 1990 feet, and 3 on 1 below. A wide berm below Elevation 1870 was constructed in front of the dam with material excavated from the core trench and with waste material from other excavations. This berm has a dual purpose: to serve as a cofferdam, and to stabilize the foundation upon which the upstream rockfill is resting. To avoid re-dressing costs, the upstream rockfill was built on natural slopes with intermediate berms every 30 feet.

In general, filter zone thicknesses, due to the high cost of processing the materials, were designed to be as thin as possible while still functioning properly. The minimum practical thicknesses, however, were often limited by construction placement methods.

Filter zones, classified as small rock filter, coarse filter, and fine filter were placed on both downstream and upstream sides of the core. The materials vary in size from 10 inches to 200 mesh, with smooth transition of gradation from rockfill to the impervious core. The downstream small rock rock filter zone extends from overburden to top of the dam, and was designed to have a minimum thickness of 15 feet. The coarse filter zone on the downstream side, designed to be 17 feet thick, was terminated at the junction with the overburden since the natural river material from that elevation down to bedrock was considered to be an equivalent substitute. The downstream fine filter zone extends from bedrock to top of the dam and has a five-foot thickness, except at the abutments where the thickness was increased to 10 feet.

The upstream small rock filter zone was designed to be nine feet thick and to cover the upstream coarse filter down to the elevation of the natural overburden material. The corase filter zone has a minimum thickness of 13 feet on the upstream side and extends down to bedrock, although the portion below the fine filter is considered only to be a buffer zone for the core material. The fine filter zone was designed to be five feet thick across the entire width, extending down to Elevation 1870 feet.

The impervious core, which extends from bedrock to the top of the dam, was designed with a downstream slope of 1.37 on 1, and an upstream slope of

1.5 on 1 above Elevation 1800 feet, and 1.8 on 1 below. The width of the contact surface between the impervious core and bedrock at Elevation 1690 feet is about 100 feet, which is considered to be conservatively sufficient to safely withstand the 387 feet of head. The width of the core at Elevation 2076.9 feet, the control elevation of the core, is 20.8 feet, and the thickness normal to the average slope at the same elevation is approximately 12 feet. The maximum amount of seepage through the core is expected to be less than one cubic foot per second.

Stability Analysis and Settlement

Stability analyses of the main dam embankment were made for both the upstream and downstream slopes. Potential failure plane on the upstream side of the embankment was considered to be along the impervious core, which is the normal assumption for an inclined core rockfill dam. Using the slideblock method, the analysis showed a minimum factor of safety of 1.38 for the most critical condition, which would occur during initial filling, when the reservoir water surface reaches Elevation 1930 feet.

There is normally little doubt about the stability of the downstream slope of a rockfill dam of ordinary height built on sound rock foundation. At Brownlee, however, due to the overburden, the stability of the downstream slope was analyzed along Elevation 1750 feet, the contact surface between the overburden material and the rockfill. With an internal friction angle of 28 degrees, the minimum factor of safety was found to be 1.53 for the critical condition, which was with the tailwater at Elevation 1827 feet.

The dam is slightly curved in an upstream direction on a 4505-foot radius. The maximum offset from the base line is 55 feet. Settlement will occur both downward and in a downstream direction. Due to the curve and the valley cross section, the impervious core upon settling will be squeezed, thus eliminating the possibility of cracks due to stretching.

Based upon past experience, a well-constructed rockfill dam could be expected to have a vertical settlement of less than one percent of its height. However, for appearance as well as for settlement, a maximum camber of seven feet was provided.

There is relatively little information in engineering literature about the settlement of rockfill dams, particularly those built on deep overburden. Previous settlement measurements have indicated that in the process of settling, some of the large rocks in the rockfill section have a tendency to rotate and thus cast doubt on the validity of the measurements. Monuments have been set in the Brownlee embankment to measure this rotation as well as the normal settlement of rockfill and core.

Construction

Careful consideration of equipment, time, and personnel, and a thorough economic analysis of the work processes involved, resulted in establishment of the following construction schedule:

<u>November, 1955 to September, 1956</u>

Mobilize
Build Access Road and Bridges
Excavate Powerhouse and Diversion Tunnel Headings

Drive Diversion Tunnel
Commence Rockfill

October, 1956 to February, 1957

Close Cofferdams and Divert River
Excavate Foundation Core Trench and Toe Trench
Grout Foundation
Construct Core and Filter Zones to Elevation 1800$\pm$ Feet
Construct Downstream Rockfill to Elevation 1810$\pm$ Feet in the River Channel Portion
Start Abutment Rockfill Placement

March, 1957 to June, 1957

Divert Over the Partially Completed Dam
Continue Placing of Rockfill from Both Abutments as Excavated Material Becomes Available

July, 1957 to February, 1958

Perform Necessary Clean-Up of the Embankment
Raise the Core and Fill to Where the Flood Will Pass Through Diversion Tunnel and Outlets

March, 1958 to June, 1958

Complete Embankment.

Wet Season of 1956

After mobilizing the necessary equipment, work progressed concurrently on several phases of construction. A bridge across the Snake River and access roads interconnecting the project features were rapidly completed. One of the roads connected the two portals of the diversion tunnel to allow use of one truck-mounted jumbo for drilling both tunnel headings.

The 2500 feet of 38 by 42-foot modified horseshoe section diversion tunnel was holed through in three months. Rock proved to be very good and only 300 feet of the tunnel required supports.

Rock from excavation for the powerhouse and muck from tunnel drilling was used to construct a section of the upstream berm of the embankment part way across the river. This preliminary embankment forced the river to flow on the Oregon side, where the water scoured out objectionable materials in the embankment foundation area.

Dry Season of 1956

When closure of the upstream cofferdam was completed and water diverted through the tunnel, the foundation area was unwatered and excavation of the core and toe trenches commenced on a 24-hour a day schedule. Unwatering was accomplished by two 20,000 gpm pumps, each installed in a pipe frame which could be picked up by a dragline and relocated as the excavation proceeded. Nearly 400,000 cubic yards of core trench excavation in the river channel were completed in less than three months. Equipment used to accomplish this excavation feat included two shovels of 3 and 3-1/2-cubic yard

capacity, one 2-1/2-cubic yard dragline, one 5-1/2-yard Manitowoc dragline, and Euclids of 17-cubic yard capacity.

Foundation drilling and grouting was also accomplished in the same three-month period. The operations followed immediately after excavation of the core trench and proceeded from both abutments toward mid-river. Two systems of foundation grouting were used. One was a blanket grouting over the entire core trench area, and the other was a cutoff grout curtain.

The area grouting consisted of drilling holes on 10-foot centers a minimum of 10 feet deep. These holes were grouted under pressure of about 20 psi minimum. The grout "take" was very small in holes in the core trench area on the right portion of the river channel. Except near Dike No. 4, where a few holes took 12 to 34 cubic feet and one hole took 272 cubic feet, the amount of "take" was approximately that required to fill the drill holes. In the left portion of the river channel, the presence of seams and cracks in the foundation rock resulted in greater "take," varying from a few cubic feet to a maximum of 254 cubic feet in one hole with some loss to surface leaks.

Cutoff grout curtain holes were drilled to a depth of 75 feet on 20-foot centers with intermediate holes drilled to 150 feet. These holes were grouted under pressures up to a maximum of 300 psi. When the "take" was high, additional holes were drilled at 5 or 10-foot centers between the original holes. As in the area grouting, the right portion of the foundation took less grout than the left. The maximum "take" was 324.5 cubic feet in one 75-foot hole. A total of 58,961 linear feet of grout holes were drilled and grouted in the core trench with an average "take" of about 0.62 sacks of cement per linear foot of hole.

Core excavation revealed a fault near the center of the river channel. At the upstream end, the fault was 8 to 12 feet wide and filled with sand, gravel, and large boulders. The fault opening tapered to the thickness of a seam both in depth and in a downstream direction. Based on the grouting record, it was found that the fault was very tight.

While the foundation treatment was in progress, the borrow areas for impervious and filter materials were being prepared. One D-8 bulldozer was used to strip from two to five feet of soil overburden which was unsuitable for use in the core. Impervious clay core material was excavated from the borrow area by a 2-1/2-cubic yard shovel and loaded on bottom-dump Euclids. One shovel was able to efficiently supply loads to 15 Euclids of 17-cubic yard capacity. Round trip from borrow area to core area and back required about 45 to 60 minutes per truck. Shovel production varied from 100 to 150 cubic yards per actual working hour, but the average production per shift-hour, including moving, repair and idle time, ranged from 80 to 90 percent efficiency.

For the fine filter material, the main equipment used was the blending plant consisting of a 24" belt conveyor fed by bulldozers with fine river sand and pit-run filter material in a 30 - 70% proportion. A 1-1/2" screen separated the blended filter from the coarse material and the products were conveyed by belts to stock piles.

Immediately after foundation grouting in the core area was completed, placement of the clay core commenced. Artesian water entering the trench was encountered in lesser quantities than previously predicted since some of it was intercepted by the diversion tunnel, and some was deterred by specially drilled and grouted holes near the left abutment. However, since it was impossible to stop all the seepage to enable placement of the clay core on dry

rock in the deepest portion of the core trench, which was the river fault area previously mentioned, a pile of clean gravels was used to fill the low spot with two 24-inch pipes, 8 to 10 feet long, embedded vertically. The gravel was capped with a concrete slab, and clay core material placed on top while seepage water was pumped through the 24-inch pipes. When the core reached a sufficient height, the gravel was grouted with cement through one of the pipes, forcing the water out through the other.

The impervious core was placed under rigid control, especially near the bedrock strata where it will be subjected to the highest water pressures. The "in-place" core material had a moisture content slightly higher than optimum. Tests of the material indicated a permeability less than 10 feet per year, and plasticity index higher than 10.

Initial compaction of the core against the rock foundation was effected by pneumatic hand tampers. When the working area became large enough, sheepsfoot rollers with specially designed wedge-shaped feet were used to compact the impervious material in about 6-inch layers. Usually, 12 to 16 passes were required to obtain the specified 90% modified AASHO optimum density.

The severe cold weather during the period might have caused progress to fall behind the tight construction schedule except for the fact that heat loss in the core material was small during hauling and placing operations. After hauling, spreading, and rolling the material, ice did not form in the completed layer, provided that the succeeding layer was immediately spread.

A typical temperature record in degrees Fahrenheit of one of the coldest days was reported as follows:

Air temperature	10
Clay temperature as hauled to the embankment	45
Temperature at surface of the compacted fill	31
Temperature at one-inch depth	32
Temperature at two inches depth	33
Temperature at three inches depth	34

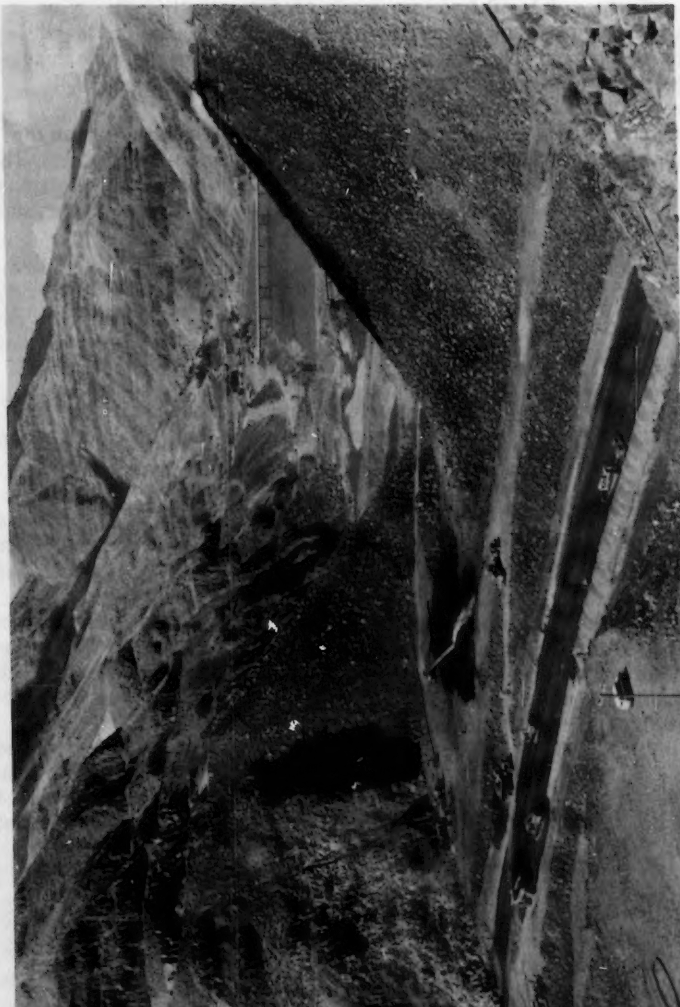
If the operation was suspended too long, however, formation of needle ice in the clay could be observed. Therefore, clay chunks of suspicious nature were removed by hand when they could not readily be broken with a hand shovel.

More than 100,000 cubic yards of impervious material were placed in the core trench in less than two months. Progress on the core placement was stopped by the spring floods with the top at Elevation 1800 feet. The maximum daily placement progress (three shifts) amounted to more than 3,000 cubic yards of compacted impervious fill. (See Photograph 1)

Coarse filter materials were end or bottom-dumped, and fine filter materials were end-dumped against the coarse filters or the impervious core. The filters were compacted in the same operation, which resulted in a smooth transition from fine to coarse zones. Satisfactory compaction required at least four passes of the Euclids loaded with 17 cubic yards of material.

The toe trench was excavated to bedrock, and then backfilled with large rocks of which 50 percent were one-half cubic yard or larger in size.

Large selected rock, weighing one-half ton and over, will also be used to form a protective layer on the face of the upstream rockfill section.



PHOTOGRAPH I: BROWNLEE DAM DURING CONSTRUCTION

View looking downstream showing impervious core and filter zones and rockfill.

Wet Season of 1957

When the construction schedule was being formulated, one of the major considerations was river diversion. At the damsite, the low water season usually starts in late June or early July and lasts five to eight months. During this season, the flow varies from 8,000 to 20,000 cfs, and seldom exceeds 24,000 cfs. The first floods start as early as December, with normal high flood flows usually occurring in April or June with peak discharges from 50,000 to 70,000 cfs.

Computations disclosed that one 38-foot unlined horseshoe tunnel could pass the low water season flows, but two tunnels would be required to carry the high water season flows. Careful studies indicated that with a relatively small risk, diversion could be accomplished by constructing only one tunnel and allowing flood waters in excess of the capacity of the tunnel to flow over the partially completed fill. For the protection of the core area, it was planned to cover the core and filter areas with two to three feet of pit-run coarse filter materials and top with seven to eight feet of rockfill.

On February 24, 1957, however, intense rainfall on the barren hills immediately upstream from the site caused a flood peak to occur at the site very suddenly. All available men in camp were summoned to help move equipment out of danger in case the river overtopped the upstream cofferdam.

Before water reached the top of the upstream cofferdam, which was at Elevation 1835 feet, the road from the damsite to Robinette was seriously endangered by rising water. To save this road, the cofferdam was opened and water allowed to overflow the fill before the intended protection could be provided for the core. (Photograph 3) At the time of overtopping, the downstream rockfill was at Elevation 1850$\pm$ feet near the abutments, and at Elevation 1809$\pm$ feet for a width of about 250 feet in the central portion. The impervious core and filter zones in the core trench were approximately at Elevation 1800$\pm$ feet.

The flood reached a peak of 70,000 cfs and an estimated 40,000 to 50,000 cfs flowed over the fill. There was approximately 20 feet of drop in the water surface in the 2,000-foot distance between cofferdams.

Following this first peak, the normal flood season set in and lasted about four months. During that time, the river flow ranged from 30,000 to 70,000 cubic feet per second, with the later flow occurring a second time on May 23, 1957.

Dry Season of 1957

By the middle of June, the river flow had receded to 20,000 cfs and the upstream cofferdam was again closed. Clean-up and inspection of the core zone, which began immediately after closure, revealed that the diversion scheme had been sound.

There was no apparent loss of rock in the main rockfill zones. The core area, which had been purposely kept lower than the rock area, was almost completely filled with mud and rocks to the elevation of the rockfill. Most of the material deposited in the core area apparently came from the upstream cofferdam.

Removal of this deposited material revealed that the original compacted impervious core had not been appreciably disturbed by the river waters. To ascertain if any internal change had occurred, a test pit five feet deep was excavated. Material removed from this pit showed no change in moisture content below the top few inches. Even in the top few inches, the increase

was surprisingly slight. Only an insignificant amount of original core had to be removed before construction could be resumed. Diversion over the partially completed rockfill dam was considered to be a complete success.

Although the major portion of the rock in the embankment was placed during the 1957 dry season, placement actually proceeded throughout the entire construction period. Care was taken to meet the construction schedule and still maintain a balance between rock excavation and fill. Thus construction proceeded with no appreciable delays or stockpiling of rock. This record was maintained not only for the dry season of 1957, but for the entire construction period.

Along the right abutment, the first lift of rock was dumped at about Elevation 1900 feet. Embankment proceeded simultaneously toward the downstream toe and mid-river. As the trucks end-dumped their loads over the edge of the embankment, a monitor mounted on the outer end of a skid which stuck out over the edge of the embankment sluiced the rock down the slope using a 2-3/4 inch nozzle and water pressure ranging from 75 to 100 pounds per square inch. The monitor supplied about four times as much water by volume as the rock being sluiced.

On the Oregon side, rock placement began on the gravel bar away from the left abutment. Placement reached an elevation of 1850 feet and then progressed toward the abutment. By this means, a good contact was made between the sluiced rock and the steep abutment slopes. When the spillway excavation reached top production, another lift was started from this abutment at Elevation 1900 feet, and then a final lift at Elevation 2050 feet. Rock placement proceeded from both abutments, at equal elevations, gradually closing the gap between the slopes in the middle.

The amount of smaller-sized rock obtained from the spillway excavation was larger than had been expected. The excess 6-inch and smaller rocks were placed and compacted in the center portion of the dam. Analysis of the compacted small rocks indicated that stability of the fill was not impaired. In addition, the amount of settlement is expected to be no greater than in the sluiced rock portion. The small rocks were similar to the small-rock filter zone material and were placed and compacted in the same manner; that is, in 18-inch layers compacted by not less than three passes of 50-ton pneumatic rollers. After each lift was compacted, the surface was sluiced. All of the small rock was covered with not less than 40 feet of large sluiced rock at the downstream face of the dam.

Placement of the downstream rockfill progressed far ahead of core and upstream rockfill placement. (See Photograph 2)

Excavated rock which was considered unsuitable for use in the downstream rockfill because of small size or too high a percentage of fines, was placed in a portion of upstream rockfill zone. This area which was designed for use of this material did not require all the material and the remainder was placed in the upstream berm. The large rock portion of the upstream rockfill was sluiced in place by monitors.

Rockfill Production

During the peak rockfill operations, 6 monitors and 48 Euclids were in use. Maximum production was about 30,000 yards in two 9-hour shifts.

Practically all rockfill material was obtained from excavation for other features. Prior to June 1957, there was limited space for placement of the

PHOTOGRAPH 2: ROCKFILL CONSTRUCTION

View looking upstream showing dumped rockfill being placed and sluiced.

PHOTOGRAPH 3: DIVERSION OF RIVER OVER PARTLY COMPLETED DAM

View looking downstream from the Idaho abutment showing the flood channel thru the rockfill.

excavated rock in the dam. Total volume placed to that date was about 2,300,000 cubic yards, of which 800,000 cubic yards were from powerhouse and tailrace excavations, 700,000 from power intake, and 800,000 cubic yards from spillway. After clean-up from river immersion was accomplished, construction of both the core and rockfill proceeded at top speed.

The major embankment quantities placed, in cubic yards, were as follows:

Upstream cofferdam	150,000
Downstream cofferdam	150,000
Upstream rockfill	1,582,000
Downstream rockfill	3,976,000
Filter zones	468,000
Impervious core	449,000
Upstream berm to El. 1870 feet	706,000
Total Fill	7,481,000

Rock excavation was accomplished with drills of the percussion type and bit sizes varying from 2-1/2 inches to 4 inches. Lifts varied from 15 to 25 feet with holes from 5 feet to 10 feet apart on centers. The production averages were as follows:

	Drilling	Powder Rate
Powerhouse	0.72 L.F./cy	0.48 lb/cy
Power Intake	0.70 L.F./cy	0.61 lb/cy
Spillway	0.66 L.F./cy	0.55 lb/cy

The amount of rock excavation in cubic yards obtained from required excavation was as follows:

Powerhouse	477,000
Tailrace	440,000
Spillway	1,690,000
Power Intake	1,400,000
Core Trench	377,000
Toe Trench	403,000
Diversion Tunnel	156,000
Diversion Inlet Channel	143,000
Diversion Outlet Channel	183,000
Total Excavation	5,269,000

CONCLUSION

1. Brownlee Rockfill Dam will be completed in July 1958. Unit cost for rock placed in the dam is estimated to be about half the cost for similar construction 10 to 15 years ago. Improvements in earth-and-rock-hauling equipment in recent years, combined with modern construction methods, have made rockfill-type dams much more competitive with other types.

2. The large quantities of rock required in the embankment made the heavy excavation at other features of the project feasible and resulted in substantial savings of the individual feature costs. Careful site adaption, arrangement of the project and scheduling of operations were required to achieve the overall project economy.

3. Rockfill dams may be safely supported on consolidated riverbed materials of sand, gravel, and boulders provided foundation explorations have been performed to ascertain that there are no extensive silt of fine sand layers. Careful analysis of rockfill embankments should be made to satisfy the stability requirement.

4. When normal rock excavations provide a considerable quantity of small rocks, economics can be effected without loss of structural strength by placing them in the center of the embankment with sufficient compaction.

5. Passing flood water over a partially completed rockfill dam is entirely feasible. Before adopting such a diversion scheme, detailed hydraulic analyses should be prepared to determine probable flow characteristics over the embankment.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: KAJAKAI CENTRAL CORE DAM, AFGHANISTAN[a]

Glenn F. Sudman,[1] M. ASCE
(Proc. Paper 1735)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

This paper describes the design and construction of the rockfill dam embankment portion of Kajakai Project. This dam, in Central Afghanistan, was economically constructed despite its geographic isolation. The feat required maximum use of locally obtainable materials and complete training of a native work force.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1735 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at Meeting of ASCE, Portland, Ore., June 1958.

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INTRODUCTION

The Royal Government of Afghanistan has initiated an improvement program for revitalizing an area which, in past history, was referred to as the "Garden Spot" of Asia. This area, which in modern times is called "Helmand River Basin", occupies approximately 107,000 square miles of Southern Afghanistan. The major lifeline of the basin is the Helmand River. As shown on Figure 1, the river flows in a 600-mile circuitous path from the Hindu Khush spur of the Himalaya mountains to the Seistan Sink on the Iranian border.

The precipitous, barren slopes of the upper basin, combined with a climate predominated by winter rainfall and dry hot summers, have resulted in extreme annual variations of river flow. The fertile valley lands have been subjected to alternate periods of drought and flooding. In order to alleviate this serious condition, the combined forces of International Engineering Company, Inc. and Morrison-Knudsen Afghanistan, Inc. were commissioned to design and construct a dam and reservoir for control and development of the Helmand River.

Reconnaissance of the river revealed the best damsite was near the village of Kajakai, about 75 air miles northwest of Kandahar. At this site, the ancient Helmand River had cut through beds of limestone to form a short, steep-walled gorge. Upstream from the gorge, the river valley widens and is encompassed by low rolling hills. The natural topography thus provided an excellent site where a considerable volume of reservoir storage could be economically developed.

Preliminary investigations of streamflow data, irrigation demands, downstream flow requirements, and power development potentialities resulted in the selection of a multi-purpose project. Through economic analyses led to the decision that the most feasible project development would be attained by construction in two stages.

The first stage, as shown on Figure 2, included a main embankment, side saddle spillway, and two diversion tunnels. Various combinations of dam and spillway arrangements were studied. These resulted in the selection of a dam as shown on Figure 3, with top at Elevation 1050 meters. The spillway crest was set at Elevation 1033.5 meters. Spillway discharges were uncontrolled. Reservoir capacity to the top of the spillway crest would then be approximately 1,470,000 acre-feet. This amount was considered adequate for water requirements, mostly irrigation, under the present scope of land development.

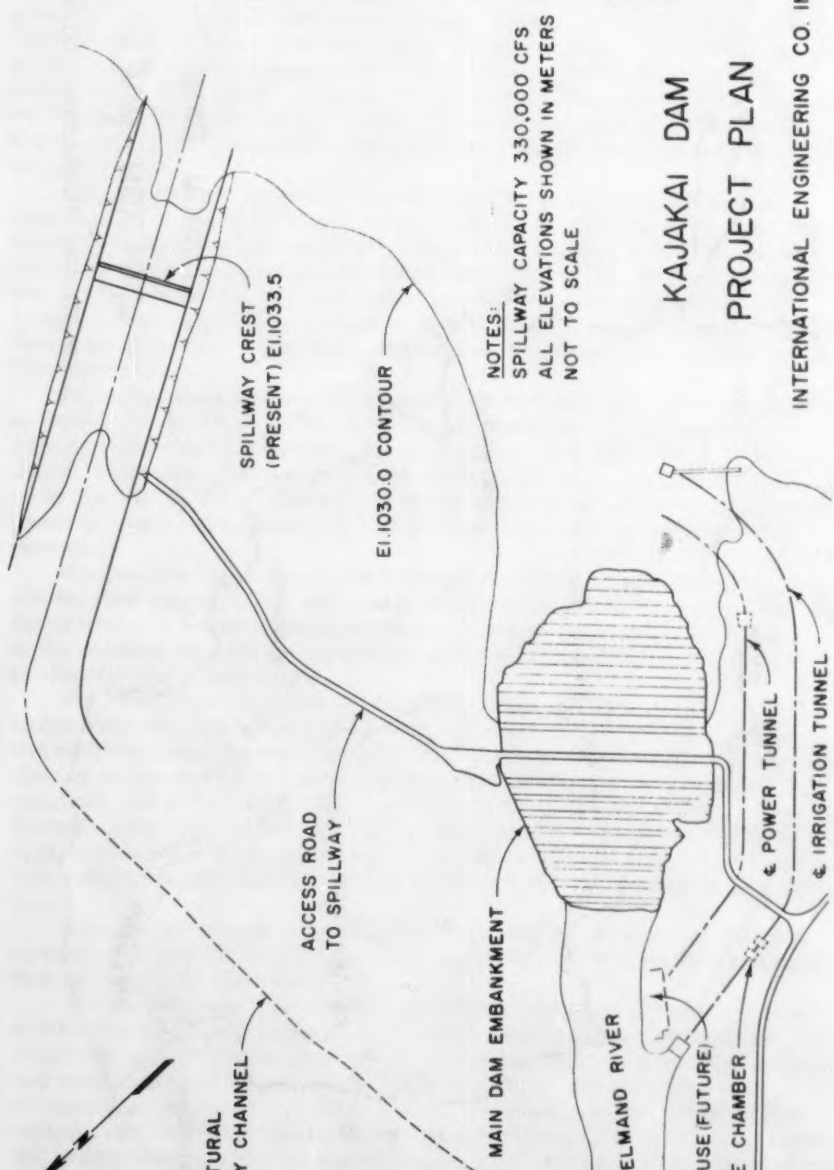
After diversion use, one of the tunnels would be fitted for irrigation releases. The other tunnel would be plugged until power requirements justified its further development.

The second stage can be constructed when irrigation demands increase to where more water is required, or when a market for power develops. This stage will consist primarily of the installation of gates in the spillway channel and construction of the remaining power facilities. Allowing five meters of freeboard on the dam crest would set the top of the gates at Elevation 1045 meters, and would provide an additional 830,000 acre-feet of storage, making the total reservoir capacity approximately 2,300,000 acre-feet. This amount would be adequate for irrigating almost 500,000 acres, an area slightly greater than any land development project contemplated in the foreseeable future. In addition, irrigation releases and the developed head would provide energy for a potential firm power load of 60,000 kilowatts. Selection of the actual type and size of generating units has been deferred until the need for power is established.

FIG. I

KAJAKAI DAM
LOCATION MAP
INTERNATIONAL ENGINEERING CO. INC.

FIG. 2


KAJAKAI DAM
PROJECT PLAN
INTERNATIONAL ENGINEERING CO. INC.
NOTES:
SPILLWAY CAPACITY 330,000 CFS
ALL ELEVATIONS SHOWN IN METERS
NOT TO SCALE
SPILLWAY CREST
(PRESENT) El.1033.5
El.1030.0 CONTOUR
ACCESS ROAD
TO SPILLWAY
MAIN DAM EMBANKMENT
NATURAL
SPILLWAY CHANNEL
HELMAND RIVER
POWER HOUSE (FUTURE)
ROTOVALVE CHAMBER
℄ POWER TUNNEL
℄ IRRIGATION TUNNEL

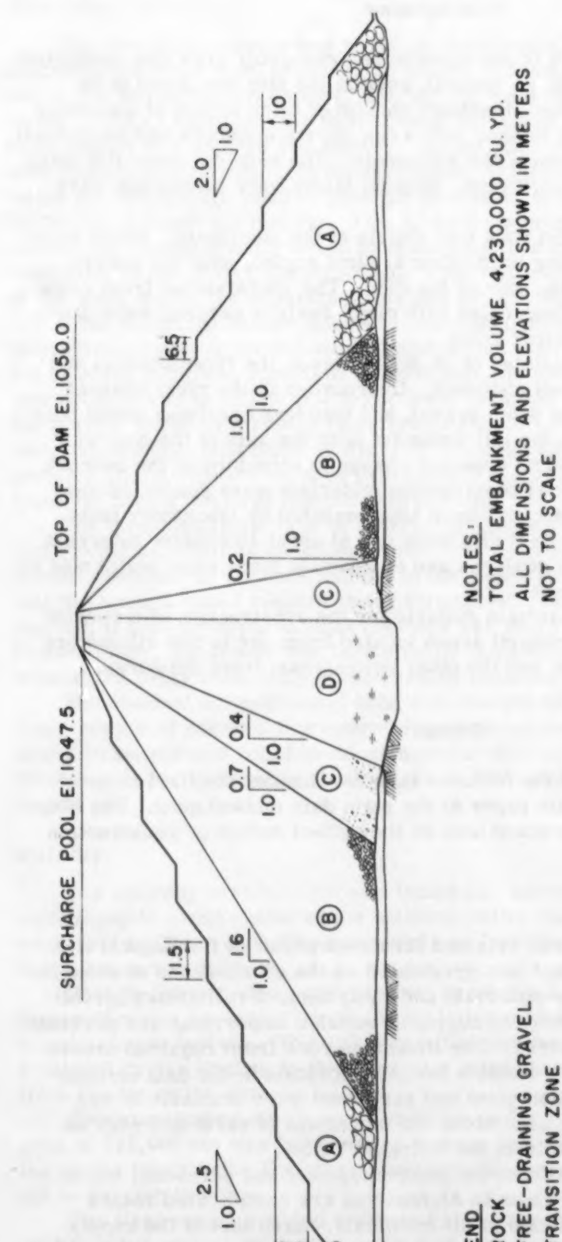


FIG. 3

KAJAKAI DAM

EMBANKMENT SECTION

INTERNATIONAL ENGINEERING CO. INC.

Investigations

Geological examination of the damsite and reservoir area was conducted during the summer of 1950. In general, rock at the site was found to be massive dolomitic limestone. Beddings consist of thick layers of limestone near river level, becoming thinner with a few cherty members and occasional shale interbeds near the tops of the abutments. The beds dip generally about 6 to 8 degrees toward the northwest. Several faults were visible but were not considered serious.

Extensive jointing of the rock was visible at the abutments. Joints were in two directions intersecting each other at right angles, with the pattern forming oblique angles to the axis of the dam. The joints varied from close to open and were more or less filled with clay. Surface caverns were disclosed on some of the principal joints.

Foundation borings consisted of 23 drill holes in the river channel and one horizontal hole in the left abutment. Overburden in the river channel consisted of about 30 feet of sand, gravel, and boulders overlying sound limestone bedrock. The hole in the left abutment near the axis of the dam was driven 210 feet and revealed no apparent change in structure of the bedrock.

Investigations of potential construction materials were conducted near the site during 1948 and 1949, and were supplemented by laboratory tests made in the United States. Test pits were dug at about 100-meter intervals in three areas. Mechanical analyses and compaction tests were performed on materials from these test pits.

Ample quantities of materials suitable for the construction of a rockfill dam were found in two borrow-pit areas located from one to two kilometers from the dam, one upstream and the other downstream from the gorge.

Design

Discussion of design of the features completed under the first stage of construction is limited in this paper to the main dam embankment. The other features of the dam are discussed only as they effect design or construction of the main embankment.

Selection of Type of Dam

A rockfill type of dam was selected for development of the Kajakai site. Design of rockfill embankment was predicated on the efficient use of economically available construction materials and equipment. Preliminary investigations had shown that an adequate supply of suitable impervious and pervious materials were available nearby. The limestone rock from required excavations was found to be entirely suitable for incorporation in the dam embankment. In addition, construction plant and personnel were available in this remote country, having been assembled for movement of earth and rock on a large scale for other projects in the Helmand Valley.

Conversely, cement and concrete plant equipment and personnel would have to be imported. Supply lines to Afghanistan are complicated routes and require several re-handlings of all materials. Each link of the supply line is subject to interruptions, which in turn interrupt construction schedules dependent upon outside supplies. The long line and interruptions result in high construction cost of any structure requiring large amounts of imported bulk items such as cement.

Diversion

The diversion scheme was based on construction of a low embankment upstream from the dam, of sufficient height to divert low water flow through one of the two diversion tunnels. During the eight-month period before high runoff could be expected, the downstream portion of the rockfill section was to be raised to Elevation 985 meters. At the start of the high water season, both tunnels will be available for diversion. Water at Elevation 985 meters would provide enough head to pass the maximum diversion design flood of 70,000 cfs through the tunnels. Two 32-foot diameter concrete-lined horseshoe-shaped tunnels would have the required capacity and would provide flexibility of use during construction. This was especially true for completion of the work within the separate bores and for adaptability to final water use.

If the diversion design flood was exceeded, minor overtopping of the downstream rockfill section could occur without damage, as a protective coating of heavy rock was specified. In addition, the embankment upstream of this rockfill was to follow at a lower level to obtain smaller velocities over the portion of the dam having finer grained materials. This was in agreement with construction schedules as the time required for foundation clean-up and grouting in the impervious area precluded placement of a large amount of material during the first eight-month period.

The target estimate type of contract permitted final designs to be easily and quickly adjusted as conditions were disclosed during construction. The jointed condition of the rock had led to the conclusion in the original design that the tunnels would require lining throughout. After the tunnels were started, it became apparent that the rock was much better than anticipated. Consequently, the design was immediately modified to eliminate the lining without the delay frequently occasioned by contract re-negotiations.

The nominal unlined tunnel size was changed to 34 feet. At the same time, review of construction operation schedules revealed that the top of the downstream rockfill could be constructed to Elevation 990 meters. The diversion capacity resulting from these modifications was computed to be 63,000 cfs.

Spillway

The spillway arrangement was based on: advantageous use of the topography to avoid paving of the spillway outlet channel, and to balance the amount of spillway excavation as nearly as feasible with the amount required in the embankment.

Rock in the selected outlet channel is adequate to withstand the erosive forces of water for many years. Some lining may be eventually required as a maintenance feature. The spillway outlet channel excavation terminates in a natural ravine leading to the river at a point about 300 meters downstream from the toe of the dam.

Two hypothetical floods were used for spillway design. One flood with a peak of 318,000 cfs was to be routed through the reservoir without encroaching on the freeboard. The other flood with 50 percent greater magnitude was not to overtop the dam.

The first-stage, ungated spillway has a capacity of 330,000 cfs and a usable surcharge of more than 1,000,000 acre-feet at normal freeboard level. A check review of hydrological information accumulated in the intervening period will be made to verify the accuracy of the design floods at the time the gates are added.

Main Embankment

The main embankment, as shown on Figure 3, has a crest length of 275 meters and is approximately 100 meters high as measured from bedrock to top of dam. The total embankment volume is approximately 3,200,000 cubic meters.

Complete removal of overburden material was required only in the impervious core area. In the remaining embankment area, removal of only unstable materials containing clay or silt was required. Naturally-deposited river sand and gravel, which was left in place, was to be scarified to a depth of six inches and re-compacted prior to placing the embankment.

On the abutments, bedrock was exposed over much of the area. Consequently, treatment consisted of removal of overhangs, sloping of vertical faces and elimination of sudden changes in slope. This abutment sloping was required to provide a wedging action as the fill consolidated, and to eliminate the possibility of the fill "hanging up" and cracking at the abutment contact during the consolidation process.

The zoning within the dam was predicated on the most efficient use of the available materials. Preliminary investigations had indicated that definite quantities of materials, varying from impervious to previous, were available. From the tunnel and spillway designs, estimates were made of the quantities of rock available from these excavations. An early decision was to keep the core and transition zones at a minimum thickness consistent with prevention of percolation through the dam and the efficient use of placing and compaction equipment. Several factors influenced this decision: relative availability of materials, cost-in-place of the fill, and the comparative lack of structural strength of these types of material. Further decisions limited the use of rock to that available from required excavation. Quarried rock would be more costly for an equivalent effect on dam stability than pervious materials from borrow pits. As the actual quantity of suitable rock available from required excavations would probably differ from the estimated amount, zoning limits between gravel and rock materials were made flexible.

As finally designed, the dam consisted of:

a. An impervious core of compacted material composed of a mixture of alluvial sand, silt, and clay.
b. Zones of compacted transition material upstream and downstream of the impervious core.
c. Pervious sections of compacted free-draining gravel upstream and downstream.
d. Rockfill sections, upstream and downstream, comprising the outer shells of the dam.

The embankment slopes were determined by stability analyses, using conservative strength factors for the various materials. The factors of safety, as determined from these analyses, under all conditions of loading, exceeded the minimum values generally considered permissible.

Embankment Materials

Specifications

Specifications for field control were issued to ensure that the minimum requirements utilized in the stability analysis were satisfied.

PHOTOGRAPH NO. 4 KAJAKAI DAM EXCAVATION OF CORE TRENCH, START OF IMPERVIOUS CORE FILL, AND STRIPPED LEFT ABUTMENT

PHOTOGRAPH NO. 5 KAJAKAI DAM EXCAVATION OF CORE TRENCH – LOOKING TOWARDS LEFT ABUTMENT

The impervious material was specified to be a well-graded mixture of sand, gravel, clay, and silt possessing low to medium plasticity with not more than 40 percent gravel and no stones larger than 4 inches in diameter. The permeability after compaction was not to exceed 1.0×10^{-4} centimeters per second (103.5 feet per year). Minimum shear strengths, as determined from triaxial shear tests of saturated, unconsolidated, undrained representative samples, were stipulated to be equivalent to an angle of internal friction of 17 degrees and an apparent cohesion of 1.5 kilograms per square centimeter. Compaction was to be 98 percent of standard density (ASTM Test D-698), with a moisture content of between two and four percent on the dry side of optimum.

Grading of the transition zones from impervious on the inside to pervious on the outside, with shear strength equal to or greater than the impervious zones, was specified. Special selection of material was required for a 20-meter width adjacent to each abutment, which was to be highly impervious, and for a layer of free-draining, highly pervious material overlying the foundation in the downstream portion of the dam.

The gravel zone material was specified to be free-draining with a permeability after compaction of not less than 400×10^{-4} centimeters per second (41,413 feet per year). Compaction was required to a dry density of 95 percent of the maximum dry density obtained in a laboratory test. An angle of internal friction of 34 degrees was specified to agree with the value utilized in design.

The rockfill was to consist of sound limestone obtained from excavations, sluiced into place with the volume of water twice the volume of rock. Special selection of large, more resistant rock was required on the downstream face below Elevation 990 meters and on the upstream face between Elevation 1010 meters and Elevation 1050 meters. Lifts were specified to be dumped in 10-meter heights with outside slopes equal to the natural angle of repose. Berms of appropriate widths were to be provided at the top of each lift to give average slopes equal to the design slope. Variation in the slope of the contacts between gravel and rockfill zones was stipulated to suit the quantity of suitable rockfill material found available from the required excavations.

Field Control

No unusual problems were encountered in the placing and control of materials. The borrow pits were found to be characterized by a gradual transition from fine impervious silts to coarse river-run gravels, with no clearly defined lines of demarcation. Careful field control using permeability limits as a guide was exercised so that permeability in the transition zone between the impermeable core and the pervious gravel shell was increased gradually. Tests of the in-place materials showed that shear strengths greatly exceeded minimum requirements. The relative ratio of permeabilities was also substantially higher than required. The moisture content of the materials in the borrow pit was very low. However, this was economically corrected by diking borrow areas and flooding them until the moisture content was satisfactory. Compaction to the specified densities was readily accomplished by sheepsfoot rollers and by pneumatic hand tampers.

The advantage of providing flexibility in the design of the embankment was demonstrated during construction. The faulted zone in the spillway proved to be larger than anticipated, with the result that the amount of rock suitable for use in the embankment was less than originally estimated. As

construction of the embankment proceeded, the substitution in the upper portion of the dam of large gravel and cobbles in lieu of a portion of the exacavated rock was approved after re-examination of stability.

A second example of flexibility was in the use of impervious materials from borrow areas. The placing of impervious material in the embankment was delayed by fault treatment in the core trench area. Because of the delay, the amount of materials used from the upstream borrow pit was less than originally scheduled. As a consequence, more extensive use of the downstream borrow area was required. This possibility had been recognized and the existence of an adequate reserve had been determined beforehand.

Foundation Treatment

The pattern of intersecting joints, solution channels and faults crossing the site necessitated extensive and careful treatment to ensure that leakage through the rock would be minimized. A very careful study in the field, accompanied by surveys of the main open joints and water testing through drilled holes, resulted in the adoption of a final program consisting of three general operations.

Shafts were excavated in the faulted areas for satisfactory cutoff and backfilled with concrete. Surface exposures of joints which would be subjected to water pressure were cleaned out and filled with gunite or concrete plugs. Holes were then drilled diagonally to intercept the greatest number of joints and then grouted.

The joints in some places were found to be filled with clay which would tend to prevent the flow of grout, but which in time might be removed by flow of water. For this reason, it was felt that the sealing of surface exposures that might otherwise permit the upstream entry of water into the pattern of intersecting joints was of prime importance. Due to the directional pattern of the jointing system, special attention was directed to the left abutment where the treatment of surface exposures was continued for some distance upstream. The segments of the tunnels which would be subjected to reservoir pressure were similarly treated. In some locations on the left abutment and on the tunnel walls, the gunite seal blanketed extensive areas of exposed rock surface. Where grouting was performed in the vicinity of capped exposures, the surface treatment prevented the escape of grout except through vents left specifically for that purpose.

The foundation excavation disclosed three nearly vertical faults cutting diagonally across the core; one in the right abutment, one near the toe of the right abutment, and the other near the middle of the river.

The right abutment faults were filled with tight clay and gouge. These faults were four and six feet wide and were excavated to about 70 feet and backfilled with concrete.

Treatment of the fault in the center of the river became a major operation. The faulted zone was about 16 feet wide and filled with fractured but solid rock, interspersed with bands of dense calcite laminated with cement gouge and clay. The shaft, keyed into solid rock was located near the upper side of the core and had a length slightly greater than the width of the zone. Excavation of the shaft in this fault required blasting. However upon exposure after blasting, the clay slaked and shrank, loosening the walls causing cave-ins and rockfalls.

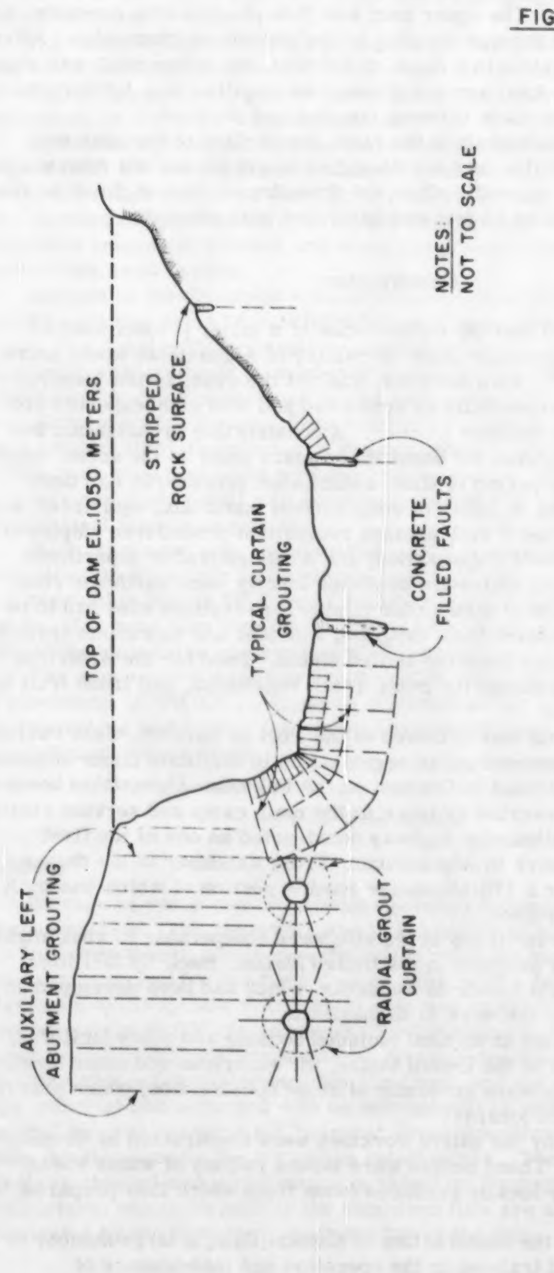


FIG.4

KAJAKAI DAM

LONGITUDINAL SECTION

INTERNATIONAL ENGINEERING CO. INC.

The excavation of this shaft proceeded very slowly and proved to be a construction bottleneck. After the shaft reached a depth of 60 feet, work became very hazardous. The upper part was then plugged with concrete. An access opening was left through the plug to the underlying excavation. After completion of the excavation to a depth of 100 feet, the entire shaft was plugged with concrete. The total amount of concrete required was 1,800 cubic yards. A grout seal was made between the plug and the rock.

In addition to the main shaft in the fault, the surface of the zone was cleaned out to a depth of five feet for its entire length across the core and backfilled with concrete. A second shaft at the downstream edge of the core zone was excavated to a depth of 15 feet and backfilled with concrete.

Construction

It might be expected that the construction of a major project such as Kajakai Dam in the historically isolated country of Afghanistan would prove to be unusually difficult. This however, was not the case, as the construction was accomplished essentially as scheduled and with no unexpected problems resulting from the isolated location. A construction organization had been operative in Afghanistan for about three years prior to the actual start on Kajakai Dam. In this period of time, established procedures had been developed for maintaining supplies of construction materials, equipment, and spare parts. These included well-planned requisition procedures employed by an efficient procurement organization, and a skilled traffic department.

The only construction materials available locally were earth and rock, except for a small amount of poor grade lumber. Everything else had to be imported. Except for cement from Belgium, and fuel and lubricants from the Middle East, imports were from the United States. Food for the American personnel was imported except for meat, fresh vegetables, and fresh fruit in season.

All imported material was received at the Port of Karachi, West Pakistan, where the Constructor maintained an organization to facilitate trans-shipment. The next step was by railroad to Chaman on the Pakistan-Afghanistan border. The supplies were then carried by truck to the main camp and service center at Kandahar over a 70-kilometer highway constructed as one of the first phases of development work in Afghanistan. From Kandahar to the damsite, truck transport was over a 170-kilometer road, a portion of which was built as part of the Kajakai Project.

Storage and service facilities at the site were comparable to what might be maintained at similar projects in the United States. Back-up facilities were provided at the main center in Kandahar, which had been developed as headquarters for most of the work in Afghanistan.

The construction camp at Kajakai included housing and mess facilities, equal to living standards in the United States, for American and other foreign personnel. The buildings were generally of stone construction, which proved to be satisfactory for this locality.

Facilities required by the native workmen were constructed by themselves in separate compounds. These people were issued rations of whole wheat and rice as well as other locally produced foods from which they prepared their own meals.

Prior to and during the construction of Kajakai Dam, a large number of native workmen received training in the operation and maintenance of

construction equipment, and in other contruction skills. They proved very apt pupils and no difficulty was encountered in developing an adequate number of skilled workmen.

The majority of the foreign personnel on this project were in administrative, technical, and supervisory categories. At the height of construction activity, the force at the jobsite consisted of 1850 native workmen and 69 foreigners, of whom 54 were Americans.

In May 1950, facing-off of the diversion tunnels was started. By August 1951, the low initial cofferdam fill was completed and low season flow diverted through one of the tunnels. At the end of September, the site had been unwatered, most of the overburden excavated in the core trench, the foundation treatment started, and some progress made on construction of the downstream rock section.

Success of the diversion scheme required that the planned construction schedule placing rates be carefully followed during the first phase of the work. The construction operations proved to be well planned and adequate equipment was available to meet this schedule. No difficulty was encountered except for the delay occasioned by the excavation of the shaft in the fault zone under the core trench. This fortunately caused no interference with the most critical item, the placing of the downstream rockfill. After the extent of this fault zone treatment was determined and the effect on the construction schedule realized, a wall was constructed around the area to permit the embankment placing to proceed.

Rock placement was concentrated on the downstream rockfill to ensure completion to Elevation 990 meters prior to the spring floods. This elevation was reached in March 1952, and flooding occurred at the end of the same month. The peak flow was 29,000 cfs and the water surface upstream from the fill reached a maximum elevation of 970 meters.

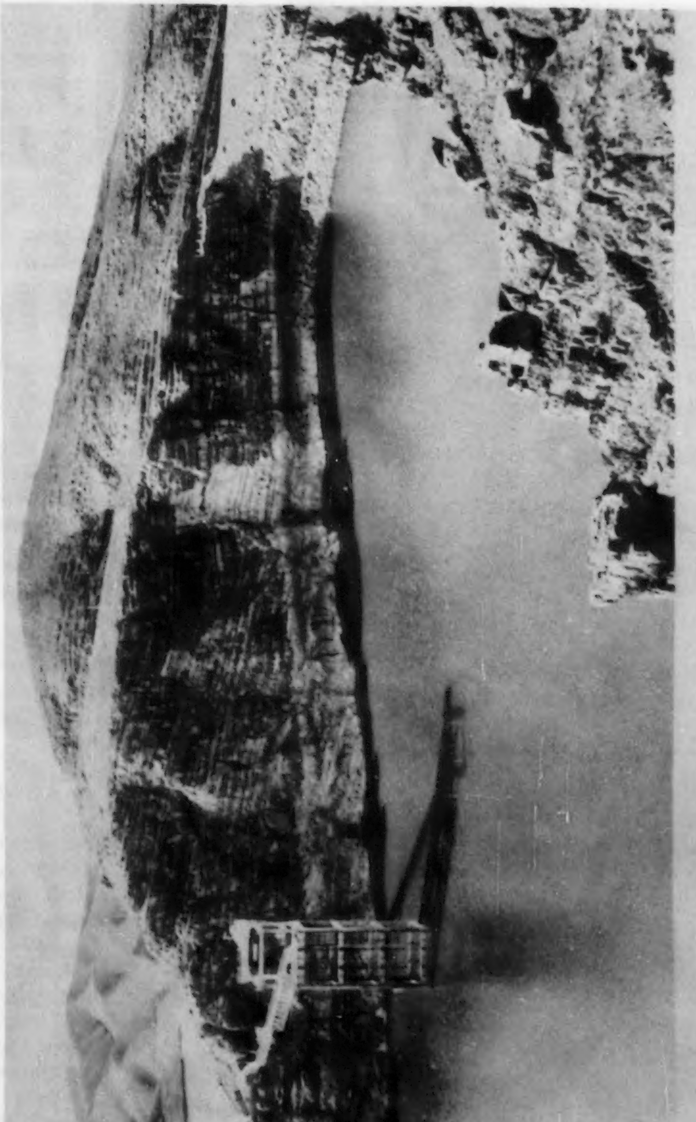
The dam embankment was completed in late 1952, with an average monthly placement of 390,000 cubic yards of material during the height of activity. Photographs 1 and 2 show the completed embankment, looking upstream and downstream, respectively.

Performance

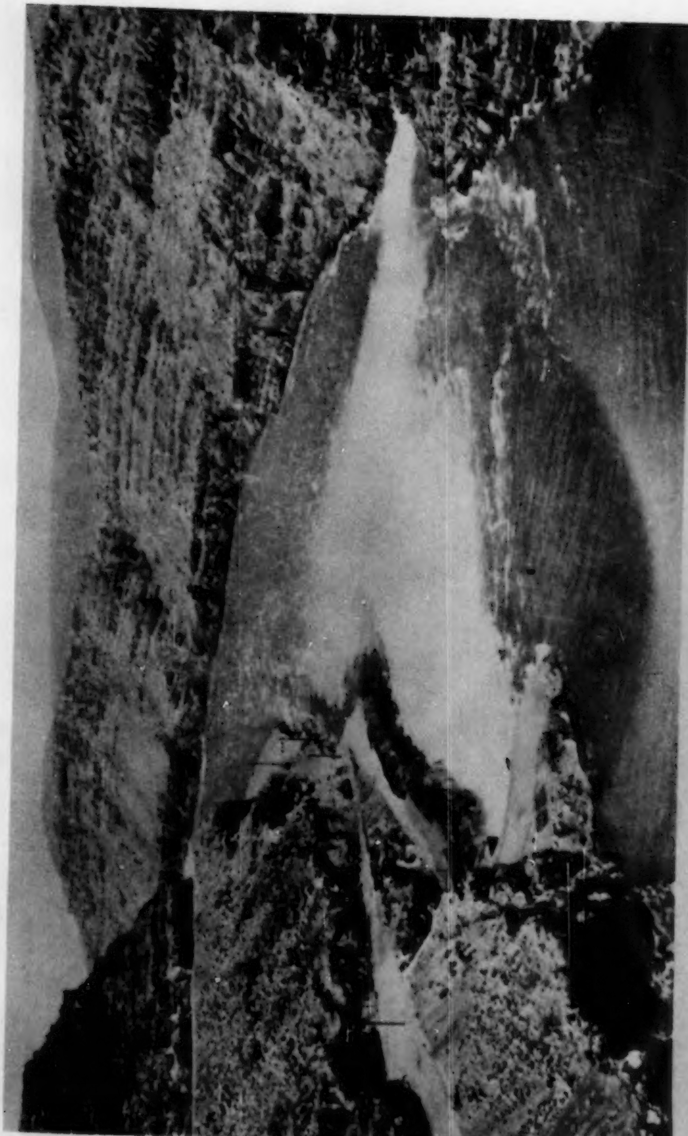
Spillway

Storage of water commenced immediately following substantial completion of construction. The reservoir was filled nearly to spillway crest the first season, even though continuous irrigation releases were made. Flow over the spillway occurred for the first time in 1954, with a peak flow of 24,000 cfs. Maximum spillway flow recorded to date was in 1957, when a peak of approximately 60,000 cfs occurred. Performance of the spillway during these flows has been entirely satisfactory. Overburden was left in the natural spillway channel downstream from the crest, and the expected erosion of this material has occurred with no detrimental effects. Discharge from the natural channel reenters the Helmand River downstream from the toe of the dam, nearly opposite the irrigation outlet works. The combined flow from the spillway channel and outlet works is shown on Photograph 3. As shown in the picture, waves formed by the combined flow are almost completely dissipated by the time they reach the toe of the dam.

PHOTOGRAPH NO. 1 KAJAKAI DAM MAIN DAM EMBANKMENT AND OUTLET WORKS LOOKING UPSTREAM



PHOTOGRAPH NO. 2 KAJAKAI DAM MAIN DAM EMBANKMENT AND IRRIGATION INTAKE TOWER LOOKING DOWNSTREAM



PHOTOGRAPH NO. 3 KAJAKAI DAM COMBINED DISCHARGE OF SPILLWAY AND OUTLET WORKS LOOKING DOWNSTREAM FROM TOP OF DAM

Effectiveness of Seepage Control

Effectiveness of the seepage control was noted by visual observation of the embankment. The fault treatment, grouting, and core compaction method have apparently been very successful. In addition to visual observation, piezometers installed during the embankment construction have been read at frequent intervals since the original filling of the dam in 1953, and the readings carefully studied. Pore pressures and phreatic lines derived from the readings have been plotted and are presented on Figures 5 and 6. In general, the readings indicate that pore pressures within the impervious core follow the rise and fall in reservoir water surface elevation. There was no appreciable lag in response of pore pressure to pool variations. In all cases, even for the example taken two months after completion of the core, the phreatic line was fully developed to the stage expected for the steady state condition.

Settlement

Settlement and horizontal displacement was observed from readings of the triangulated range established at Kajakai. Between June 1954 and December 1957, the average settlement of the crest was approximately two inches, while the horizontal displacement averaged three-fourths of an inch in a downstream direction. Both figures are considered to be indicative of negligible movement.

Cost-Benefit

It would be very difficult to place an exact monetary value on the actual benefits derived from construction of Kajakai Dam and Reservoir. A few construction costs as related to various project features are presented, however, as an aid for individual evaluation of potential benefits.

The actual final cost of the dam approached $12,500.000. The total quantity of earth and rock in the main embankment was about 4,200,000 cubic yards; therefore, the overall cost including all structures and appurtenant work constructed in the first stage was about three dollars per cubic yard. Placing the actual earth and rockfill portions of the embankment had a final cost of about $1,000,000, resulting in a unit cost of about 25 cents per cubic yard. Based on the actual final cost, and a reservoir capacity of 1,470,000 acre-feet, the first-stage cost was about $8.50 per acre-foot.

The cost of the dam, with respect to irrigable lands, can be evaluated on the basis of land development studies. As of 1957, these studies indicated that approximately 220,000 acres were being irrigated by Helmand River waters downstream from Kajakai Dam. The annual interest and amortization of the expenditures for Kajakai Dam applied to the presently irrigated lands amounts to less than 50 cents per acre-foot.

The regulation provided by Kajakai Dam has not only enabled new acreage to be brought under irrigation, but has given a firmer and more usable supply of water to lands previously irrigated. When the lands below the reservoir are developed to the extent that full use is made of the water stored in the first stage of Kajakai Reservoir, the cost of the dam will result in an annual average cost of 36 cents per acre-foot of water.

These figures do not include the power potential at Kajakai which may eventually have an ultimate installed capacity of 120,000 kilowatts and flood

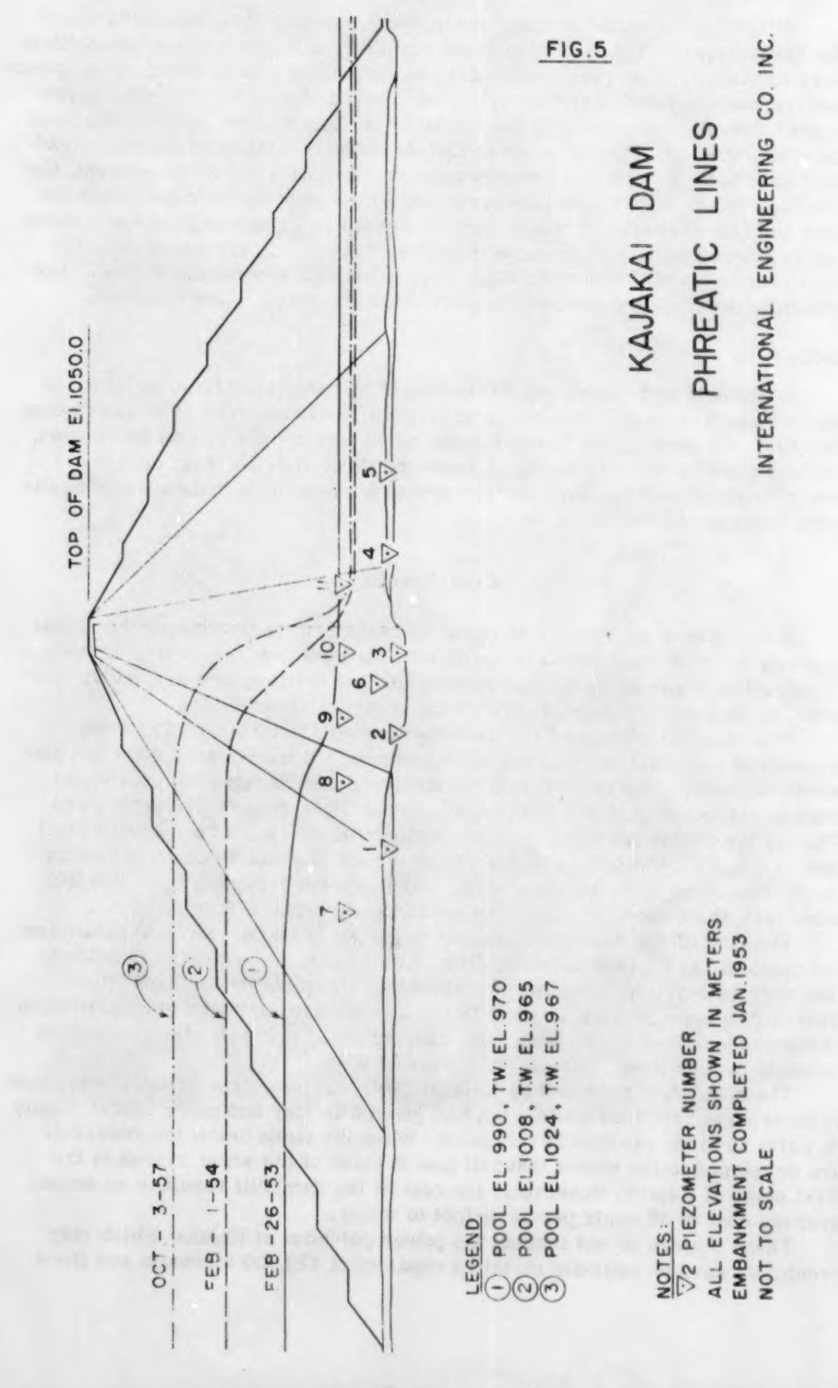


FIG.5

KAJAKAI DAM
PHREATIC LINES
INTERNATIONAL ENGINEERING CO. INC.

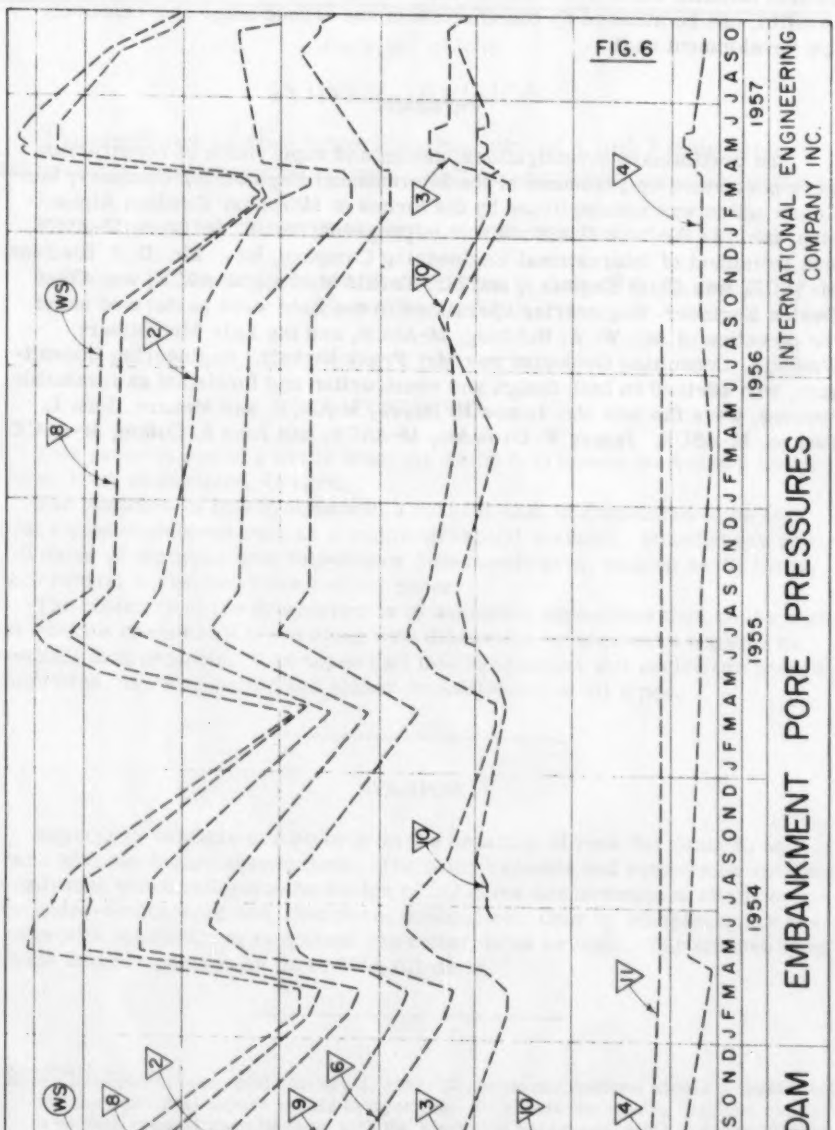


FIG. 6
KAJAKAI DAM EMBANKMENT PORE PRESSURES
INTERNATIONAL ENGINEERING COMPANY INC.
PRESSURE ELEVATION IN METERS
1035
1030
1020
1010
1000
990
980
970
965
1953
1954
1955
1956
1957
WS
8
2
9
6
3
10
4
5
11

control benefits which are considerable. These, as well as further water use benefits, can be attained by construction of the second stage at a relatively low development cost.

Personnel

The preliminary investigations, design and supervision of construction were performed by personnel of the International Engineering Company, Inc. Construction was accomplished by the forces of Morrison-Knudsen Afghanistan, Inc. At the time this work was in progress, Mr. C. P. Dunn, M-ASCE, was President of International Engineering Company, Inc.; Mr. D. J. Bleifuss, M-ASCE, was Chief Engineer; and Mr. Torald Mundal, M-ASCE, was Chief Design Engineer. Engineering operations in the field were performed under the direction of Mr. W. A. Hohlweg, M-ASCE, and the Late Mr. Gilbert Waddell. Consulting Geologist was Mr. Frank Nickell. Engineering Consultants, who advised on both design and construction and furnished an invaluable service, were the late Mr. James B. Hayes, M-ASCE, and Messrs. John L. Savage, M-ASCE, James P. Growden, M-ASCE, and John S. Cotton, M-ASCE.

Paper 1736 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: PERFORMANCE OF TVA CENTRAL CORE DAMS[a]

George K. Leonard,[1] M. ASCE and Oliver H. Raine,[2] A. M. ASCE
(Proc. Paper 1736)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Especially valuable are reports on the behavior of rock fill dams to compare with the design assumptions. The many variable and sometimes unknown conditions which influence the design of fill dams are sometimes either over- or under-emphasized and, therefore, misapplied. Only by comparing the results with the design assumptions can better dams be built. The authors have made this comparison on three TVA fill dams.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1736 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at meeting of ASCE, Portland, Ore., June 1958.

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2. Senior Civ. Engr., Tennessee Valley Authority, Knoxville, Tenn.

Presentation

The central core rock fill dams as built by the Tennessee Valley Authority (Fig. A) naturally differ from those built by other owners due to varying design assumptions, available materials, and methods of construction; but generally speaking, all consist of a heavy, nearly impervious central earth core supported and stabilized by massive dumped rock fills on the upstream and downstream faces. While emphasis will be on the performance of three of TVA's dams beginning with reservoir impoundment, it seems necessary and appropriate to describe in some detail their planning, design, and construction.

Advantages of This Type of Dam

Central core rock fill dams (Fig. 1), if properly chosen, designed, and built, have many excellent qualities. Most important of these might be their durability, their permanence. Neither fire nor flood, heat nor cold affect their life in any way. In time of war, they would have a much better chance for survival than concrete dams. They are as permanent as the hills into which they abut.

Dams of this type are economical in that use is made of natural materials at the site instead of the more costly materials required in a concrete structure; design is less complicated; and construction is relatively simple.

Three of TVA's Rock Fill Dams

Three central core rock fill dams built by the Tennessee Valley Authority will be described. These are all located on the upper tributaries of the main river, Watauga and South Holston in the northeast corner of the watershed and Nottely in the southeast corner, all on rivers of the same name. The three projects serve multipurpose objectives and have considerable flood control storage as well as power installations. A 15,000-kw unit has recently been installed at Nottely to utilize the available energy from the discharge to the downstream Hiwassee project during peak operation. Fig. 1 shows cross sections of the three dams, and Figs. 7, 8 and 9 show views of the dams.

Table of Principal Features

	Nottely	Watauga	South Holston
Time of Construction	1941-1942	1947-1948	1948-1950
Crest Length - Feet	1,942	900	1,600
Maximum Height - Feet	184	318	285
Camber for Settlement - Feet	1.7	9.2	10.1
Core Slope - Upstream	1 on 1	1 on .85	1 on .75
- Downstream	1 on 1.2	1 on .85	1 on .75
Face Slope - Upstream	1 on 2	1 on 2	1 on 2.2 - 1 on 2.1
- Downstream	1 on 2	1 on 2	1 on 2
Quantities - Millions of Cu. Yds.			
Rolled Earth Fill	0.85	1.48	2.09
Rock Fill*	0.70	2.02	3.81

*Includes filter material

	Nottely	Watauga	South Holston
Nominal Top Elevation	1794	1998	1765
Maximum Design Headwater	1788	1988	1755
Normal Maximum Headwater	1780	1959	1729

The Local Materials

The earth materials available where these three tributary stream dams are located were classified as follows: Nottely, sandy clay; Watauga, clayey sand and lean clay; South Holston, medium clay. At these locations there would have been a sufficient amount of these fine soils available for construction of an all earth dam, with rather flat face slope, but at each location there was also available a large quantity of rock for the construction of a rock fill dam. After due examination, the rock fill type with central earth core proved to be more economical. This type required less total yardage, construction time was reduced, and the diversion, power and outlet tunnels were considerably shorter. Rock classifications were as follows: Nottely, quartzite with 20 per cent mica schist; Watauga, quartzite; South Holston, sandstone with some shale.

Since the rock fills furnish most of the stability in this type of dam, wide variations of materials and dimensions of the core are possible to suit available local materials and relative costs of earth and rock. There will be a difference in settlement characteristics of the compacted earth core and the dumped rock fill, but with a heavy core this difference can have no injurious effect on the watertightness of the core.

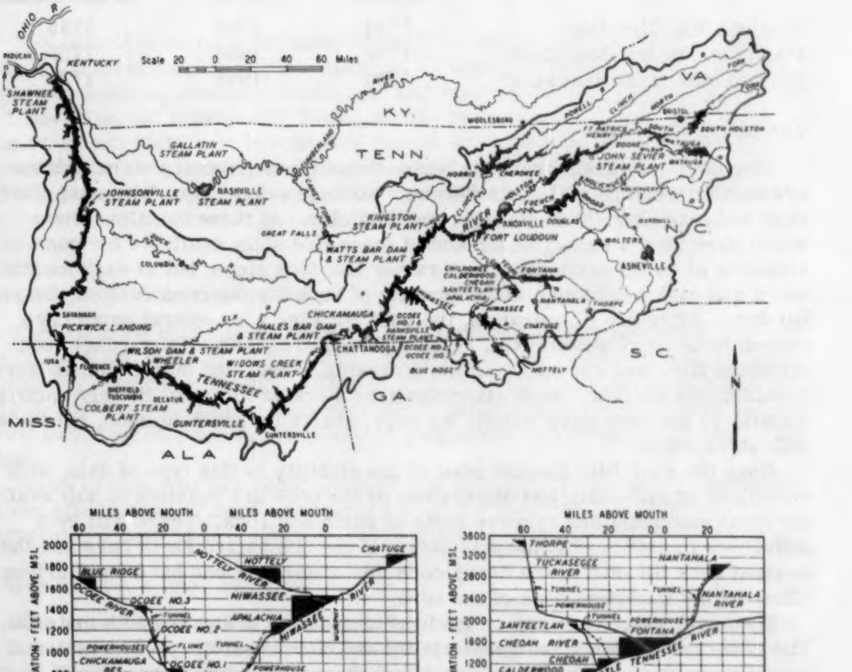
The heavy earth core can be made of almost any ordinary earth material. The governing criteria are permeability and compressibility. The material must be impervious when compacted and, therefore, must be capable of moisture control so that thorough compaction can be accomplished to provide the impermeability and to reduce the settlement in the finished dam.

Design of the Rock Fills

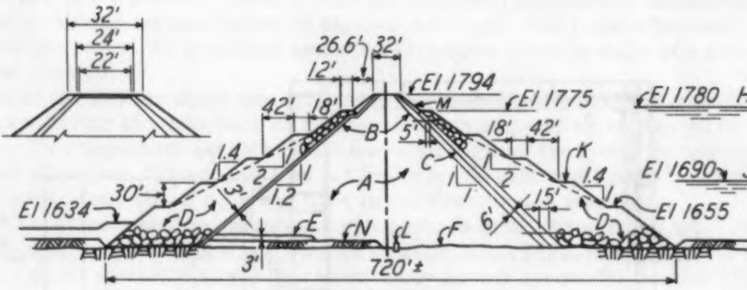
The rock fill must be made of sound rock having good compressive strength and durability against weathering. For rock fills the dry weights have been from 105 to 120 pounds per cubic foot. The quartzites, limestones, and dolomites of the region are generally acceptable, but the shales are generally unacceptable.

At South Holston Dam a high-grade sandstone was used and since the characteristics of this material were uncertain a large scale test was made to check the friction angle of dumped rock fill. Opening of the quarry at South Holston revealed that along with the sound sandstone were appreciable quantities of shale and of sandstone from which cementing material had disappeared. This uncemented sandstone had the appearance of being sound, but on handling it broke down into sand and fragments of stone. The apparatus shown in Fig. 2 was filled with 6- to 12-inch stone which was vibrated during placement. Concrete slabs were superimposed for added load, and the hydraulic jacks then applied horizontal force until actual lateral displacement of the frame and its contents took place. The tests showed an effective friction angle of 45 degrees for the sound sandstone.

When the stone was removed from the frame, several pieces were found to have actually broken, indicating the good interlocking action and crushing

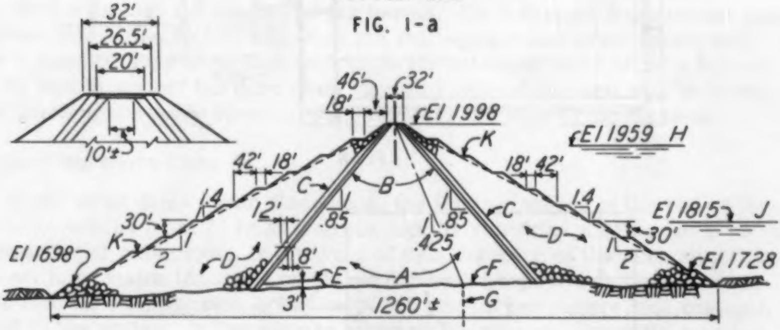


MAP AND PROFILE OF THE TENNESSEE RIVER SYSTEM



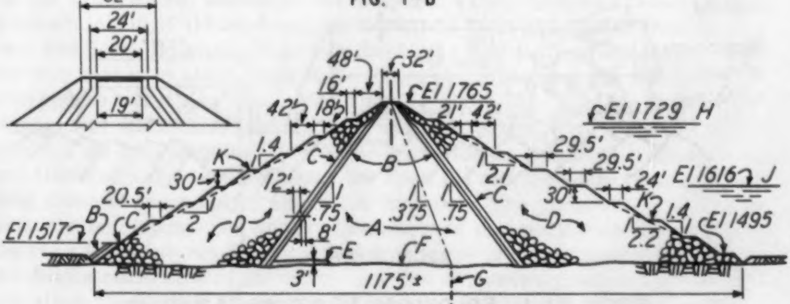
NOTTELY DAM

FIG. 1-a



WATAUGA DAM

FIG. 1-b

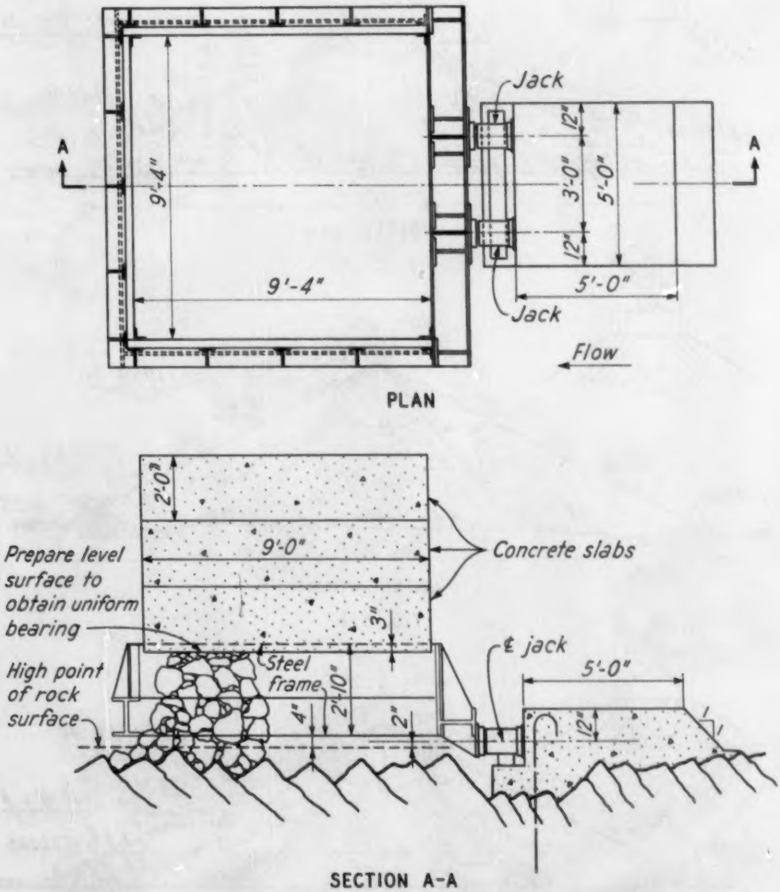


SOUTH HOLSTON DAM

FIG. 1-c

A: Impervious rolled fill.-B: Fine filter.-C: Coarse filter.-D: Quarry run rock.-E: Drain blanket.-F: Weathered rock line.-G: Grout curtain.-El: Elevation in feet above mean sea level.-H: Maximum controlled level for multipurpose operation.-J: Minimum headwater.-K: Berms spaced 30 feet vertically.-L: Cut-off wall.-M: Riprap.-N: Overburden left in place.

SECTIONS OF THREE TVA ROCKFILL DAMS



ROCK FILL FRICTION TEST APPARATUS

FIGURE 2

resistance of the pieces. Tests of both the shale and uncemented sandstone showed a friction angle of about 30 degrees for each. The tests effectively verified the use of the generally assumed 45-degree friction angle of a good dumped rock fill.

Use of the weaker stone was allowed in the upstream rock fill if mixed with enough superior stone to have an average friction angle of more than 40 degrees. To compensate for this reduction in strength of the rock, the upstream surface slope was flattened to 1 on 2.2 in the lower part and 1 on 2.1 in the upper part. Only sound rock was used in the downstream rock fill.

Both upstream and downstream surface slopes at all three dams were established at 1 on 2 except the upstream slope at South Holston as mentioned above. At all dams the slopes were broken by berms spaced 30 feet vertically, the actual slope between berms being 1 on 1.4 with the average surface slope drawn through the centers of the berms. Rock dumped from trucks on a slope not more than 30 feet high does not roll away to any great extent and takes a natural slope of about 1 on 1.4. A 30-foot-deep layer of rock fill can thus be placed against the core along the full length of the dam with no subsequent handling except to clean up the berms for the sake of appearance.

Design of the Earth Core

For the three dams under discussion, the friction angles of the core materials have been from 23 to 30 degrees and the saturated weights from 120 to 130 pounds per cubic foot. In this type of dam cohesion of the core material does not have major influence on the strength of the maximum dam section. In the heavier Nottely core, cohesion of 500 pounds per square foot was included in the design. In the steeper cores of Watauga and South Holston cohesion was ignored. The soils used actually have a cohesion of 400 to 1000 pounds per square foot, adding to the computed safety factors.

As shown in Fig. 1 the core slopes at Nottely Dam are flatter than at Watauga and South Holston. This was due to the fact that core material at Nottely was easier to obtain than rock. Then, too, the dam was built on a crash schedule and it was known that considerable core could be placed before the quarry and crushing plant were ready to produce rock fill and filter material. This suggested that the core be as wide as practical consistent with safety. Generally the core precedes the rock fill and it should be capable of standing alone at some height above the rock fill. The circumstances called for flatter core slopes. (At Nottely the height of the earth core at one time led the rock fill by as much as 100 feet.) A slope of 1 on 1.2 was established for the downstream side of the core. For the drawdown condition on the upstream slope the rock fill is dry but the earth core is saturated, providing only buoyant weight for frictional resistance. So, to maintain the desired factor of safety, the upstream rock fill was made thicker within the established 1 on 2 outer slopes by making the upstream face of the core a 1 on 1 slope.

TVA has used the slip-circle method for the design of all fill dams. For the maximum dam section at Nottely the computed slip-circle factor of safety for the downstream slope with reservoir full and for the upstream slope with complete sudden drawdown was about 1.45. In the design the internal friction of the saturated earth fill was assumed to be 26 degrees. It was later found to be 30 degrees which increased the factor of safety.

At Watauga and South Holston, rock was abundant and readily accessible at both sites, so there was no expectation that the cores would seriously lead the rock fills. The core slopes could therefore be made relatively steep. At Watauga, core material was also abundant and economically obtainable, but at South Holston earth borrow involved a longer haul and more difficult excavation. The core slopes were set at 1 on 0.85 for Watauga and 1 on 0.75 for South Holston, both upstream and downstream, with the design rock slopes at 1 on 2 for both dams.

With this core shape it was assumed that the core would be fully saturated to full pool level, making the stability of the upstream slope for the complete drawdown condition identical with that of the downstream slope for reservoir full. The core material friction angles were 30 degrees at Watauga and 23 degrees at South Holston. The computed slip-circle safety factor for the maximum dam section was 1.7 for Watauga and 1.65 for South Holston.

A foundation drain blanket was provided for the full length of Nottely Dam. It underlay somewhat more than the downstream quarter of the core. The design assumed that in this heavier core the saturation line would be lowered by the drain blanket. At Watauga and South Holston it was evident that drain blankets would be ineffective. The blankets shown on Fig. 1 for these dams cover only the exposed river bed where they might serve to intercept seepage through open rock seams and reduce uplift under the core.

Filter blankets were used between the earth and rock on both slopes to prevent the core material from being washed into the rock. Two zones were placed on all dams on the upstream side. Two similar zones were placed on the downstream side at Watauga and South Holston, but at Nottely there is only a single 3-foot-thick zone.

Construction Details

The treatment of the cutoff between dam and foundation varied to suit local conditions. At Nottely the overburden was thin except on the upper part of the left abutment. On the lower left abutment, across the channel, and on the entire right abutment all overburden was removed from the area to be covered by upstream and downstream rock fills and by the upstream half of the core. A trench was excavated to impervious rock just upstream of the center line of dam. A concrete cutoff wall in the trench was keyed 2 feet into rock and projected 4 feet up into the core. No grout curtain was required in this rock foundation. Good overburden was left in place under the downstream half of the core. On the upper left abutment, where the height of dam became less than 70 feet, two sections were used. Where the dam was 30 to 70 feet high, overburden was removed for the entire base of rock fills. Overburden was left in place under the entire core, the cutoff wall was omitted, and the upstream face of the overburden left in place was blanketed with 15 feet of rolled core material. Where the dam was less than 30 feet high, core and rock fills were founded on stripped overburden.

At both Watauga and South Holston, overburden was thin and was removed under the entire dam area except in a few areas where good overburden was left in place under the downstream half of the core. A grout curtain was provided in the rock foundation at the upstream quarter point of the core. The traditional cutoff wall was omitted. Its only function could be to make a firm connection between foundation and core in one isolated spot and increase the seepage path along the foundation plane by the total of its peripheral dimensions. With good compaction of the core on the foundation a successful

cutoff is accomplished. The presence of a cutoff wall projecting into the fill makes equipment compaction of fill near the wall difficult and calls for inefficient hand tamping. A trench 6 feet wide was excavated 2 feet deep into the rock on the line of the grout curtain and filled with concrete. This concrete cap sealed the top of the grout curtain and provided a working surface for drilling and grouting operations.

Characteristics of the foundation and fill materials were determined from preliminary investigations and conservative values of these characteristics were used in the design. Control of construction procedures to guarantee the design strength was maintained by the field engineers. All earth fill was compacted by spreading in thin layers and rolling with sheepsfoot rollers. Any overburden left in place under earth fills was well harrowed and recompacted with sheepsfoot rollers the same as the fill itself to blend fill with foundation and obliterate the junction.

Rock fills were placed by dumping from trucks. At each point of dumping one or more streams of water at 100-psi pressure from a 2-inch nozzle removed small fragments from the contact between the larger rocks and accelerated the settlement and adjustment of the fill.

Significant settlements were expected in these high dams, and so the tops of the dams were cambered to extra height to maintain future freeboard. Possible settlement was estimated from laboratory tests of the earth materials during fill construction. At South Holston, for example, the ultimate settlement was estimated to be about 18 feet and therefore the maximum section of the dam was raised 10 feet, corresponding to about the 60-year estimated settlement. Actual measured settlement at both Watauga and South Holston has been substantially less than the estimated. To avoid a sagging appearance near the top of the dam, the three upper rock berms at Watauga and South Holston were cambered successively greater amounts to build up the top camber.

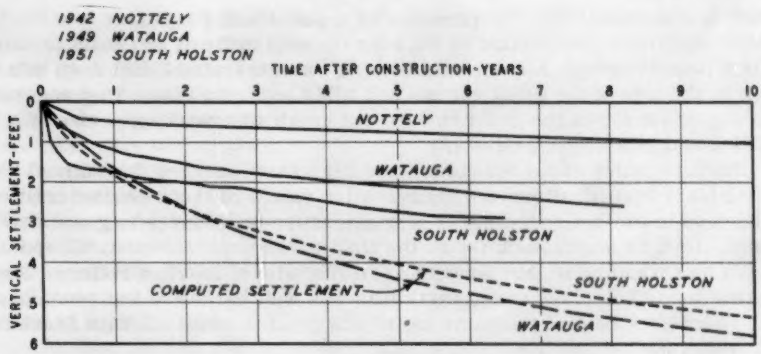
Measured Settlement of Core

Continuous records of settlement have been kept on Nottely, Watauga and South Holston Dams. Observations at Watauga and South Holston started one month, and at Nottely four months, after completion of the fill and dam closures. The reservoirs were nearly full in eight to nine months, and since initial filling they have experienced seasonal drawdowns and refillings during flood control and power storage operations.

At Nottely the fill was placed in one period; at Watauga in two periods; and at South Holston in three periods. The work periods were separated by the typical winter rainy season.

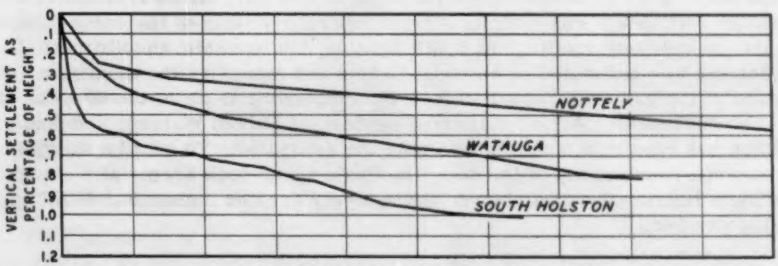
All the observation points are located on the central earth core on the maximum section and represent consolidation of the earth core under its own weight and weight of the superimposed rock shoulder. No settlement measurements were made on the rock fill itself.

Time-settlement curves for the cores of these three dams are shown in Fig. 3, plotted as direct settlement, settlement as a percentage of height, and settlement as a function of the square of the height. After ten years the top of Nottely Dam had settled 1.1 feet or 0.6 per cent of the 184-foot height. Watauga Dam in eight years settled 2.6 feet, or 0.8 per cent of its 318-foot height, and South Holston's 285-foot height was decreased in 6-1/2 years by 2.85 feet or 1.0 per cent. At age six and one-half years (South Holston's age),



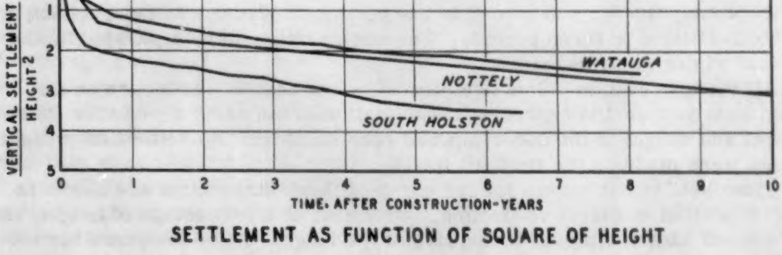
SETTLEMENT

FIG. 3-a



SETTLEMENT AS FUNCTION OF HEIGHT

FIG. 3-b



SETTLEMENT AS FUNCTION OF SQUARE OF HEIGHT

FIG. 3-c

TIME-SETTLEMENT CURVES

the percentage settlements of Nottely and Watauga were 0.46 and 0.74 compared with South Holston's 1.0 per cent.

Since consolidation of a specimen varies directly with its length and roughly with the pressure applied to it, the settlement of fills would be expected to vary approximately as the square of the height of the fills, providing the fills have the same shape and are made of materials with similar consolidation characteristics. The chart of this function shows the relative settlement curves proportionately closer together than either the direct settlement or the percentage settlement, the Nottely and Watauga curves practically coinciding.

Theoretically, the rate of settlement at South Holston and Watauga should have been about the same. At Watauga the dam is slightly higher (320 feet compared with 290 feet), producing higher compression. The rate of placing the fill was much faster at Watauga (13 months) than at South Holston (24 months). Therefore, the consolidation obtained during construction is greater at South Holston. In the detailed computation of consolidation these conflicting influences produced closely similar time-settlement curves for both dams for the first three years after completion of the fills. Thereafter the computed curve for the higher Watauga Dam showed a little greater settlement.

After the first year the curves of actual settlement of the two dams are nearly parallel, both at a slower rate than the computed. The only significant difference occurred in the first year, when South Holston experienced settlement at a much faster rate than expected. Here the top ten feet of the nominal height of the core and the superimposed 10-foot camber were compacted by truck and tractor rather than by sheepsfoot rollers. It is suspected that this contributed materially to the rapid early settlement. Fig. 3-a shows comparative computed and actual settlement curves.

The vertical settlement curves for Watauga and South Holston seem to show some response to reservoir fluctuation, as evidenced by the local steepening of the curve at the end of each of the earlier years when readings were taken with sufficient frequency to disclose the effect. (Figs. 3-a and 3-b.)

Cracks in Top of Dam

Unequal settlement of the central core and the rock fills has resulted in considerable cracking, with the resultant maintenance expense, along the shoulders of the core at all three dams although it is much less at South Holston than at Watauga and Nottely. In each case the rock fills settle more than the cores.

Progressive settlement along both shoulders for the full length of the roadway at Watauga has been noticeable for several years. Cracking on each side of the pavement (see Figs. 5 and 6) has been filled with asphalt several times. These cracks seem to follow roughly the edges of the earth core beneath the pavement. In 1957 there was a vertical displacement of up to four inches in a 50-foot section on the downstream side near the right end. This larger crack was enlarged and backfilled with clay and surfaced with hot mixed asphaltic concrete. A shorter area on the left end on the upstream side required similar treatment. The greatest settlement appears to have occurred at the extreme left end on the upstream side at the edge of the shoulder.

In contrast, at South Holston the roadway is cracked at only one short section on the downstream side at the left end where a roadway fill crosses the core to join the top of the dam. Widening the top of the core, which was

done for construction convenience, was beneficial. A comparison of the top details of the cores at Watauga and South Holston is shown in Fig. 1. There is little, if any, change in the relative elevation of the top of the roadway and the point of the shoulders at South Holston; whereas, at Watauga the shoulder in some spots is as much as six to eight inches below the center of the road.

Severe similar cracking at Nottely was observed in 1955, although prior to that time upstream settlement of the rock fill and resultant repairs had completely buried the curb under 6 to 10 inches of bituminous material. The downstream curb still retained its original appearance. In 1956 a small crack opened up along the downstream shoulder, but it was filled and today both sides seem to be stationary.

The 1955 crack extended for about 600 feet along the high part of the dam. It was in the pavement and generally was located 5 to 6 feet from the upstream edge of the embankment, but in some places it was as much as 10 to 12 feet. The crack was generally 1/4 inch or less wide, but in one or two places and over short distances the width was as much as 1-1/2 inches. Its depth exceeded 6 feet as determined by probing rods and 4-inch auger holes, the latter being stopped by rocks. There was no significant indication of any differential settlement between the edges of the crack. The pavement sloped downward between the crack and the upstream edge of the embankment.

Many ideas, some unique and possibly far-fetched, have been presented to explain the cause or causes of the cracks at these three dams. The seemingly apparent cause was expressed above—unequal settlement of the core and rock, which is influenced by differences in materials, design, and construction methods.

It is difficult to interpret the differential settlement of core and rock fill although it is generally recognized that some rapid adjustment and settlement of the rock fill is to be expected. The compaction of core material is pretty well known and settlement can be anticipated from laboratory tests. Rock fills, however, due to varying qualities of gradation, hardness, durability, roughness, and construction methods cannot be so anticipated.

As evidenced by the settlement curves (Fig. 3) good compaction was achieved in all cores. Settlement after 6-1/2 years to 10 years did not exceed 1 per cent of the height. It is believed that the low 0.6 per cent settlement at Nottely was due to the sandy soil which had a near optimum water content. At Watauga and South Holston moisture control was troublesome with the soils in the borrow pits being generally on the wet side of optimum. Aeration and drying of the soil before placing consumed much valuable time and compaction may have been hurried.

The cracking described does not endanger the stability of the dams as the friction in the rock fill and cohesion and friction in the uncracked portion of the impervious core provide adequate safety.

Deflection

Nottely Dam showed appreciable lateral deflection during reservoir filling. Upon first filling, the top of the dam at the maximum section deflected downstream nearly three inches. The deflection decreased one inch during the next drawdown and increased one-half inch on refilling.

Watauga Dam showed no significant deflection from reservoir filling. No readings have been taken at Nottely and Watauga for a number of years.

South Holston Dam has shown unusual lateral deflection of the top of the dam. A point near the center of the dam deflected upstream more than three inches during the initial loading of the dam in 1951 by the reservoir water. This deflection has varied during the subsequent seven years as shown by the curves on Fig. 4. A response of about one-half inch to reservoir changes in the early years is indicated. The upstream deflection might have been due to tilting of the dam foundation during reservoir loading. Substantial width of the valley floor at the dam site and relatively weak shale and sandstone foundation rock might tend to produce such a tilting effect. However, there is no survey information to substantiate this theory.

Seepage and Toe Drainage

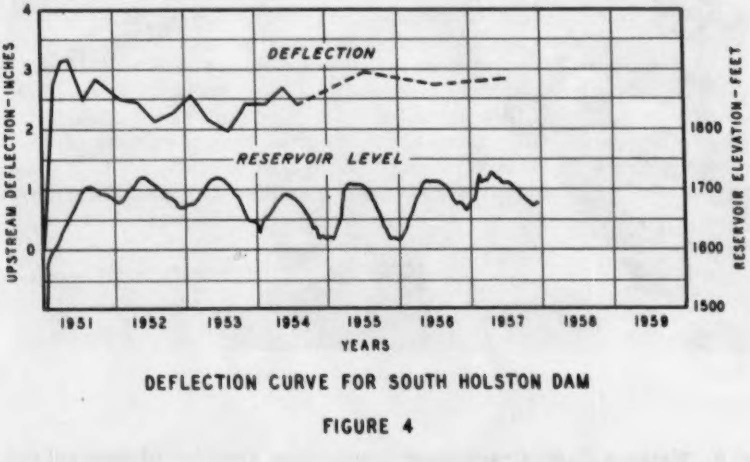
Little information is available concerning the seepage through the core and toe drainage below the three dams herein described. It must suffice to say that toe drainage has been low. At Nottely an attempt has been made to catch the drainage from the right bank and measure its flow over a weir. Hydrographs of flow compared with rainfall at Nottely during 1954 indicated that the drainage was about 6 gpm during dry periods and about 30 gpm after rainy periods.

The collection of accurate seepage and drainage flows is extremely difficult and unreliable. Rainfall, temperature, evaporation, and geology all tend to discourage the observer from placing too much reliance on his records.

Since visual inspection has consistently shown low flows, it seems not too important to worry about the actual amount of seepage.

CONCLUSION

In TVA dams up to 300 feet high the settlement of the core in six to eight years has been less than predicted from computation of simple vertical consolidation, except for vagaries in the first year or so.



DEFLECTION CURVE FOR SOUTH HOLSTON DAM

FIGURE 4

Fig. 5. Watauga Dam—Cracks forming along upstream shoulder of core

Fig. 6. Watauga Dam—Crack along downstream shoulder of core cut out, filled and sealed

The rock fill shoulders have generally experienced more settlement than the cores during the early years. Efforts should be directed toward reducing this difference in settlement as much as possible, and toward minimizing the effect of the difference. The rock in the fill should be sound and durable and it should be placed under specifications which accomplish its adjustment to stable position. If the dam is to have a paved roadway, the core at the top should be the full width of the roadway including curbs and continuous type guard rail. A stone guard fence, well adapted to these dams, was used at Watauga and South Holston. Located on the rock fill shoulder, it was made of selected large stones spaced about seven feet on centers.

Little can be deduced from the deflection experience of these dams except that they show a certain elasticity of deflection under varying reservoir load.

The spectacular in dam performance is usually associated with inadequacy or failure, of the whole or of a part. The performance of TVA's heavy central core rock fill dams has been entirely adequate, therefore not spectacular.

Fig. 8. Watauga Dam—Upstream View Near End of Construction

Fig. 9. South Holston Dam—Downstream View

Paper 1737 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: SALT SPRINGS AND LOWER BEAR RIVER CONCRETE FACE DAMS[a]

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(Proc. Paper 1737)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rickfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

The 28-year service record of Salt Springs Dam, in terms of settlement, leakage, face cracks and maintenance is presented. The troubles at Salt Springs are traced to specific construction weaknesses.

The design, construction and performance of the two Lower Bear River Dams is presented and compared to Salt Springs Dam.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1737 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at meeting of ASCE, Portland, Ore., June 1958.
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INTRODUCTION

Prior to 1925, there existed only eight rockfill dams higher than 100 feet; Morena, Strawberry and Swift Dams all being highest at about 160 feet. The eight dams were all in the United States and most in California. In 1925, a 115-foot step in height was made with the 275-foot high Dix River Dam of the Kentucky Utilities Company. This dam was a major step in the development of concrete face dams and Mr. L. F. Harza and Mr. G. W. Howson deserve particular credit. The next increase in height was made six years later in 1931 with a 53-foot step from the Dix Dam to the *Pacific Gas and Electric Company's 328-foot Salt Springs Dam. Though impervious face dams in the 200 to 300-foot range in height have since been built, Salt Springs Dam remained the world's highest until 1958, a period of 26 years. In 1958, a 27-foot step was made to the 355-foot high Paradela Dam in Portugal.

Since Salt Springs was completed in 1931, impervious core rockfill dams of various types have been developed and have been constructed to continually increasing maximum heights that presently are in the range of 400-450 feet. The economics and the high degree of safety, characteristic of rockfill dams of all types, have resulted in their increased use. The need for always higher dams of unquestionable safety makes careful study and reporting of the then existing high dams an important obligation of the engineers engaged in their design, construction and operation.

The P. G. and E. Co. serves Central and Northern California. The electric production comprises 14 thermal electric plants and 58 hydroelectric plants having a total capacity of 4,533,000 kw. The hydro plants are nearly all located in the Sierra Nevada Mountain Range which extends some 430 miles along the eastern boundary of California. Except in the northern end of the Sierras, where the mountains are of volcanic origin, the basic formation is granitic and there is little natural surface or underground storage. Precipitation is in the form of winter snow which melts in the late spring and early summer. Annual and, in some cases, cyclic storage is necessary to provide dependable hydroelectric power. To provide the storage, and to provide diversions and afterbays, P. G. and E. Co. has 106 dams of all types. With minor exceptions, these dams are utilized primarily for power purposes, but provide important benefits to irrigation and domestic water supply, flood control and recreation.

Fourteen of the 106 dams are of the impervious face rockfill type. The earliest, Fordyce Dam, dates back to 1873 and the most recent, Wishon and Courtright, are being completed in 1958. Experience with all of these dams has been very satisfactory and all of them are in service today. Development of the impervious face rockfill dam has been appropriate and particularly economic in the massive granite formations of the Sierras.

The early impervious face rockfill dam experience of P. G. and E. Co. may be considered to have culminated with completion of the 328-foot high Salt Springs Dam in 1931. Twenty-one years passed before the Company constructed the next concrete face rockfill dams, the Lower Bear River Dams in 1952. The 21 years of experience with Salt Springs as well as experience with other rockfill dams was effectively used in the design of the Lower Bear River Dams.

This paper will bring Salt Springs Dam experience up to date and will review the construction from the standpoint of its influence on the performance

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*Hereinafter referred to as P. G. and E. Co.

of the dam. The paper will also present the design, construction and performance of the Lower Bear River Dams and make comparisons with Salt Springs.

Early Pacific Gas and Electric Company Rockfill Dams

The nine impervious face rockfill dams of the Company that were constructed before Salt Springs have been termed the early dams. Data on all of P. G. and E. Company's impervious face rockfill dams is presented in Table I. All are still in service after 28 to 85 years of satisfactory performance. Experience with the six of these that are higher than 75 feet will be briefly reviewed. It was the early experience with these and other dams, together with that of Dix River Dam, that led to the adoption of a concrete face rockfill type dam for Salt Springs.

Upper Bear River Dam is now 58 years old and is of particular interest because of the steep slopes with a very thin layer of placed rock. The placed rock is one layer of hand and small derrick placed stones, and the fill is a mixture of small rock and fines. The face was timber with no concrete backing. The up and downstream slopes are the same. The 0.50:1 slope of the upper 25 feet and the 0.75:1 slope of the lower 50 feet are considerably steeper than the natural dumped rock slope of about 1.3:1. The thin layer of dry placed rock is certainly not adequate as a retaining wall. The rockfill should not have as high a shearing strength as a modern fill of dumped and sluiced large rock.

This dam and other early rockfills with steeper than natural dumped slopes have provided evidence that the shearing strength is much greater than that based on a 1.3:1 natural slope. For example, a rockfill with an infinite slope of 0.5:1 must have a shear coefficient exceeding 2.0, whereas a 1.3:1 natural slope indicates a shear coefficient of 0.77. A coefficient of 1.0 is commonly used in design of core rockfill dams. The natural dumped slope of a rockfill is not an indication of its shear coefficient.

In 1929, a dumped fill at natural (1.3:1) slope was added to the downstream face and the spillway enlarged. In 1932, the crest was raised with 5 feet of placed rock to obtain increased storage. In 1953, the deteriorated 50-year old timber was removed and a continuous gunite face with no joints was applied to the placed rock. The gunite thickness varied from 3 inches at the top to 5 inches at the base of the dam and reinforcing from one to two layers of No. 4 x 4 by No. 6 x 6 mesh.

Meadow Lake Dam is now 55 years old, and its continuous service further substantiates the stability of face slopes considerably steeper than the "angle of repose". The slopes are similar to those of Lower Bear River Dam but the placed rock is even a thinner single layer. Cutoffs are only one or two feet into granite bedrock. In 1930, a forest fire burned the timber face off the dam. A gunite face with no joints, and thickness of 2 inches at top to 4 inches at the base, was installed. One vertical crack developed in the 750 feet of crest length. It was covered by a copper joint and further opening has caused no leakage. After 27 years, the face is in good condition, even though it is at elevation 7700 and exposed all winter. Leakage is negligible. The use of thin continuous gunite without joints is considered economical and successful for both dams.

FIG. 2 THE MEETING OF 3910 LEFT ABUTMENT AND 3800 RIGHT ABUTMENT LIFTS - SALT SPRINGS DAM

FIG. 3 COMPLETED DAM WITH SPILLWAY GATES AND COPING WALL ADDED IN 1947 - SALT SPRINGS DAM

Relief Dam is now 48 years old. This 140-foot high dam has not contributed much to the technology because of the very thick section of placed rock. Though the upstream face slope is all at 0.5:1, all the rock upstream from the crest is placed. This use of placed rock was probably economical in 1910, but present costs do not justify the saving of dumped rock by use of placed rock. The 250-foot high Malpaso Dam in Peru is of nearly the same section and construction as Relief Dam. The crest of Relief is curved downstream to give a dam of minimum yardage. Downstream movement opened a vertical crack near the center of the dam. This crack was covered by a copper joint and has continued to open. This is another example of the characteristic of the impervious face rockfill dam to concentrate future movement at the initial joints or cracks. This is a very favorable characteristic.

Fordyce Dam was raised in 1927. The upstream slope of the raised dam has an 18-inch slab over 1:1 loose rockfill with nominal placed surface rocks for the lower 93 feet, and a 12 to 18-inch slab on 1:1 slope over 4 to 6-foot thick placed rock for the upper 47 feet of the raise. The successful service and small settlement of this 140-foot dam, with 1:1 upstream slope and nominal placed rock gives further confidence in the high degree of stability characteristic of dumped rockfills.

Main Strawberry Dam is now 42 years old. The slopes of this dam are 1.2:1 upstream and natural (1.3:1) downstream. Placed rock is 4 to 8 feet thick, and the fill is dumped over about 20-foot depth of streambed material. The dam is at elevation 5620 and water and frost action has deteriorated some areas of the 9 to 15-inch concrete face to the extent that at some future time repair will be required. Gunite will probably be used. This dam, with its thin placed rock and natural or steeper slopes, was a very progressive design for one of the highest dams of its type in 1916. Morena and Swift Dams (Montana), 1912 and 1914, are of about the same height.

Bucks Creek Dam is now 30 years old and has given trouble-free performance. The slopes are conservatively flat, 1.4:1 upstream and 1.5:1 downstream. Placed rock is thin, 3 to 6 feet thick. The concrete slab has no joints and reinforcing steel is continuous. The face has developed no cracks, leakage is negligible and no maintenance has been required in its 30-year life to date.

Summary

These six early impervious face rockfill dams, of 75 to 140-foot heights and ages of 30 to 85 years, have given excellent service with nominal, if any, maintenance except for replacing timber face by gunite in two cases. There has never been any question as to safety. They have provided experience that has led to the confident construction of higher dams.

Engineering Significance of Salt Springs Dam

Salt Springs Dam was completed in 1931. It was the world's highest concrete face rockfill dam, 328 feet high, until the completion in 1958 of the 355-foot high Paradela Dam in Portugal. The performance of the dam has been reported in the engineering literature.[21,22] However, in reviewing the literature in the light of later experience with the Lower Bear River and other dams, it is considered that the existing literature could be misleading. It could be inferred from the literature on Salt Springs that face cracking,

rather high settlement movements, and maintenance are a problem and that they might be expected of such a dam. Actually, these troubles with Salt Springs have never in any way given concern as to safety, cost or continuous operation. The cost of maintenance has been negligible compared to the economy in the use of the concrete face rockfill dam at the Salt Springs site.

Study of the troubles at Salt Springs has led to the design of concrete face rockfill dams of both greatly improved service record and lower cost. If Salt Springs Dam were to be constructed with today's knowledge, it would be of more simple and lower cost design and, at the same time, give superior service.

Lower Bear River Dams were designed and constructed with the help of 20 years more rockfill dam experience than was available for Salt Springs and particularly with the help of the very detailed knowledge of the performance of Salt Springs Dam. The design of the Lower Bear River Dams included features intended to avoid the face troubles that occurred at Salt Springs and these were successful. However, the Lower Bear River Dam performance, with nominal cracks and leakage and small settlement, was considerably better than expected. This fact led to a careful review of Salt Springs construction. It was evident that the superior construction of the dumped fill at Bear River was the main reason for the improved performance. Comparative physical data is presented in Table II.

In advance of presenting and comparing Salt Springs and Lower Bear River Dam data, general conclusions are that:

(1) The Salt Springs face cracks can be traced to specific construction weaknesses in the dumped fill.
(2) The most significant difference in construction was in the sluicing.
(3) In addition to construction changes, minor design changes have and can tend to eliminate the Salt Springs face troubles.

Salt Springs Dam continues to give unquestionably safe and economic service. However, to the dam engineer it has made its contribution as a laboratory for pioneering high rockfill dams. Future concrete face rockfill dams should be patterned after the design and experience of more recent rockfill dams.

Construction of Salt Springs Dam

Construction began in 1928 and the dam was completed in time to store the spring runoff of 1931. The section and profile of the dam are shown on Fig. 1 and the dumping of rock is shown on the photo, Fig. 2. In 1946-47, spillway radial gates and a coping wall were installed to obtain additional storage. The photo of Fig. 3 shows the completed dam. The construction of the dam is well described and illustrated in references (1) through (20). This paper will review construction briefly and primarily from the standpoint of its influence on the performance of the dam.

Foundation and Cutoff

Streambed and Abutment.—A total of 312,000 cu. yds. of streambed sands and gravels were removed from the floor of the narrow canyon. All material, 28 to 40 feet deep, was removed upstream from the axis. Downstream from the axis 20 to 28 feet of fine material was removed and 10 to 15 feet

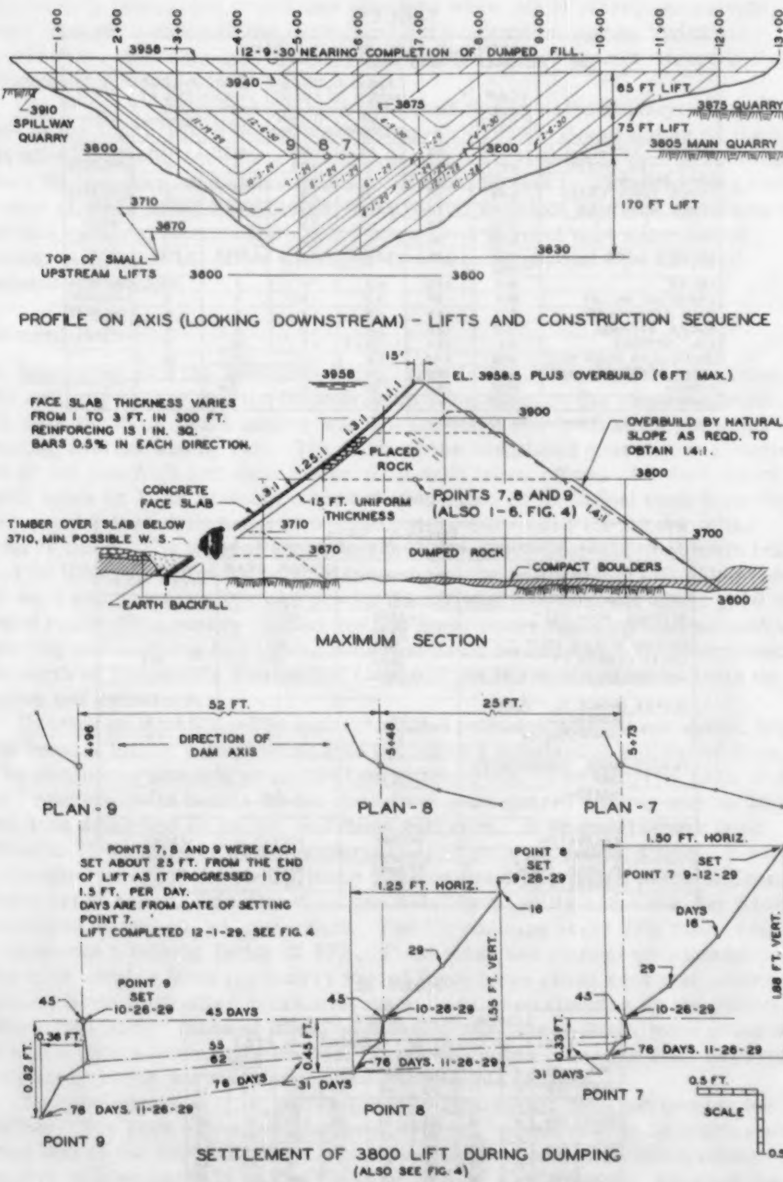


FIG. 1 PROFILE, SECTION AND SETTLEMENT OF MAIN LIFT DURING CONSTRUCTION - SALT SPRINGS DAM.

TABLE I

IMPERVIOUS FACE ROCKFILL DAMS OF THE PACIFIC GAS AND ELECTRIC COMPANY							
NAME	YEAR COMPLETED	* HEIGHT	FACE SLOPES HORIZ. TO 1 VERT.		PLACED ROCK THICKNESS FT.		TYPE OF FACE
			UP STREAM	DOWN STREAM	TOP	BOTTOM	
FORDYCE (INITIAL)	1873	93	.25	.25	LAYER	LAYER	MASONRY
" (ENLARGED)	1927	140	1.0	1.35	4	6	CONCRETE
LAKE STIRLING	1877	25	1.0	0.5			
LOWER THREE LAKES	1896	30	1.25	1.5	2	3	CONCRETE
UPPER BEAR RIVER	1900	75	0.5, 0.75	0.5, 0.75	LAYER	LAYER	WOOD
" " "	1932	80		1.3			WOOD
" " "	1953	80		1.3			GUNITE
MEADOW LAKE	1903	74	0.5, 0.75	0.5, 0.75	LAYER	LAYER	WOOD
" "	1930						GUNITE
RELIEF	1910	140	0.5	1.5	11	108	CONCRETE
CHRISTIAN VALLEY	1916	37	0.5	1.25	3	3	MASONRY
MAIN STRAWBERRY	1916	142	1.2	1.3	4	8	CONCRETE
BUCKS STORAGE	1928	128	1.4	1.5	3	6	"
SALT SPRINGS	1931	328	1.1 TO 1.4	1.4	15	15	"
LOWER BEAR RIVER NO. 1	1952	245	1.3	1.4	10	22	"
LOWER BEAR RIVER NO. 2	1952	150	1.0	1.3	10	17	"
WISHON	1958	290	1.0 TO 1.3	1.4	7.8		"
COURTRIGHT	1959	310	1.0 TO 1.3	1.4	7.8		"

* HEIGHT ABOVE FOUNDATION AT AXIS

TABLE II

PHYSICAL DATA SALT SPRINGS AND LOWER BEAR RIVER DAMS			
ITEM	SALT SPRINGS	LOWER BEAR RIVER NO. 1	LOWER BEAR RIVER NO. 2
YEAR COMPLETED	1931	1952	1952
STORAGE, ACRE FEET	142,000	49,000	
CREST ELEVATION	3958	5820	5820
HEIGHT AT AXIS, FT.	328	245	150
CREST LENGTH, FT.	1,300	960	865
SURFACE AREA OF FACE, SQ. FT.	380,000	190,000	90,000
SLOPES (HORIZ. TO 1 VERT.)			
UPSTREAM	1.1 TO 1.4	1.3	1.0
DOWNSTREAM	1.4	1.4	1.3
PLACED ROCK THICKNESS, FT.			
TOP	15	10	10
BOTTOM	15	21	17
QUANTITIES, CU. YD.			
EXCAVATION	312,000	10,000	138,000
DUMPED ROCK	2,744,000	900,000	349,000
PLACED ROCK	224,000	89,000	43,000
CONCRETE	45,000	23,000	11,500

TABLE III

LOWER BEAR RIVER DAM NO. 1 — TABLE OF CREST SETTLEMENT - IN FEET									
STA.	HT.	*	YEARS					% OF HT.	
			1ST	2ND	3RD	4TH	4YRS	1YR	4YRS
1+40	110	V	.77	.02	.02	.05	.86	.70	.78
		H	.57	.00	.07	.00	.64	.52	.58
3+20	220	V	1.10	.03	.05	.05	1.23	.50	.56
		H	.74	.06	.07	-.01	.86	.34	.39
4+40	220	V	1.03	.03	.04	.06	1.16	.46	.53
		H	.86	.02	.07	.01	.96	.39	.44
5+60	150	V	.89	.02	.05	.02	.98	.59	.65
		H	.76	.03	.02	.01	.82	.51	.55
6+80	85	V	.23	.01	.02	.03	.29	.27	.34
		H	.22	.01	.03	.00	.26	.26	.31

* V = VERTICAL COMPONENT H = HORIZONTAL

of naturally compacted gravel and boulders were left in place. Abutments were washed of nominal and occasional surface overburden by hydraulic monitoring. The dam was constructed on clean, sound granite except in the streambed downstream from the axis.

Cutoff and Grouting.—A 5-foot wide cutoff trench varied in depth from 3 feet at the top to 15-20 feet at the bottom of the dam. Below points on the cutoff where the dam exceeds a 200-foot height (el. 3770) the 2-inch grout holes were 50 feet deep, and above they varied from 50 feet to 25 feet at the crest. Below el. 3770 holes were drilled and grouted at 6-foot centers, and above at 20-foot centers, intermediate holes being used if grout take exceeded 50 sacks. A total of 161 holes averaged 42 feet in length and 27.6 sacks of cement per hole.

Dumped Rockfill

Quarrying.—Of the 2,960,000 cubic yards in the dam 880,000 came from the spillway quarry and the balance from 3 quarries on the other abutment, all near the dam. Each quarry was worked from one level since the main hauling system was by rail. The faces of the completed quarries are vertical, 60 to 180 feet high and show the vertical drill holes. Rows of 6-inch churn drill holes on 18 to 22-foot centers were located 30 to 45 feet back from the face, and 2-inch drill holes at 8-foot centers were used for lifters at the quarry floor. The largest shot (Nov. 5, 1929), forty-four 158-foot main holes and 99 lifter holes loaded with 60 tons of powder, yielded 232,000 cubic yards of solid granite. The average powder factor was 0.60 lbs. per cubic yard of solid rock. This quarry method yielded many rocks that required secondary drilling and shooting to produce rock that could be handled. The compressive strength of the granite was 15,000 to 19,000 psi, as determined by tests on cubes and cylinders.

Grading of Rockfill.—The quarry method provided many large rocks, but the heavily loaded 6-inch holes also produced a substantial amount of fines. The secondary shooting produced large clean rock. The Bucyrus 120B (4 cu. yd.) shovels could handle 25-ton rocks and main quarry hauling was by 30 cu. yd. side dump and 20 cu. yd. end dump rail cars. Rock grading was from fines to 25-ton rocks, with the average large rock exceeding 3 tons. It was estimated that half the shovel loads were on the dipper teeth which indicates many large rocks. One shovel on two 8-1/2 hour shifts and a six-day week averaged 50,000 cu. yd. per month. The fill contains about 29% voids which represents a bulking factor of 41%. Field notes and photographs indicate that the rock coming from the quarry varied from large clean rock just after a quarry blast to smaller rocks with many fines when cleaning up the quarry. When "too many" loads of fines, on cleaning the quarry floor, were going into the dam, some loads were wasted. Total waste was less than 5%. Occasionally large rocks were dumped over visible areas of fines.

The photograph, Fig. 2, shows the 3910 lift and the 3800 lift coming together. The rock of the left abutment 3910 lift is observed to be much smaller than that of the 3800 lift. This was uniformly the case since the spillway quarry was equipped to handle rocks up to only 2 cu. yd. size, whereas the right abutment quarries could handle 8 cu. yd. and larger rocks.

Lifts.—Fig. 1 illustrates the lifts used in construction. The first fill in the 3670 and 3710 lifts was to permit an early start on the cutoff and placed rock and the lifts were of minimum top width. The three main lifts, Fig. 1,

were each for the full thickness of dam. The main and most important lift of the dam was 185 feet high. Since the natural slope was about 1.30 to 1.35:1, it was necessary to construct berms beyond the 1.4:1 specified slope. Surface cleanup on the lifts was not good. The important surface was that at el. 3800 and the field engineer's report reads, "The spillway quarry 3910 lift is overlapping with no 3600 cleanup . . . an attempt is now being made to roughen the top of the 3800 lift by shooting holes 20 ft. apart and afterward sluicing fines into the larger voids at the bottom of these holes . . . a 3-inch low velocity jet moves the fines but holes 3 to 4 feet deep and 8 feet in diameter quickly silt up . . . a higher velocity jet would be desirable but is not possible . . . as the higher lifts overlapped the 3800 level it became dangerous to work on the cleanup". This is significant since the subsequent horizontal crushing failures all took place at el. 3800. The 185-foot dump from the 3800-foot level would have permitted considerable fines to accumulate in the top zone of the lift.

The 165-foot main lift was certainly a great benefit to Salt Springs Dam. The many large rock of 10 to 25 tons do considerable compaction as they slide or plow down the dumped slope. The height of lifts in the upper 199 feet or so of the dam are less important since pressures due to water and rock loading are both low.

Sluicing.—The superior performance of the Lower Bear River Dams over Salt Springs occasioned a thorough review of the Salt Springs construction records, particularly regarding sluicing. It is definite that Salt Springs sluicing was not comparable to present practice. The first 750,000 cubic yards were dumped between Feb. 1928 and July 1929 with only a 100 gpm-350-foot head-centrifugal pump and 4 and 2-inch pipes and hoses on the job.

Dumping had been on the order of 80,000 cu. yds. per month and sluice water less than 20,000 cu. yds. per month and not available at all points of dumping. A 300-gpm pump was added in July 1929. Sluicing was still recognized to be inadequate and on October 1, 1929, with 1,100,000 cu. yds. already in the dam, one 1200 and one 1500 gpm pump was installed along with 8 and 4-inch lines, and sluicing began in earnest. The 3800 lift of 1,500,000 cu. yds. was completed in Jan. 1930 and was thus only nominally sluiced. Though the last 1,900,000 cu. yds. of rock was dumped with a substantial amount of water, the method of application was not that which would be used today. The application of water was by letting hoses pour onto the top of the lift at or near the point of dumping. There were no jets to remove pockets of dirt and spalls and even with adequate volume of water the method of application would not permit the water to be fully effective. The fact that sluicing of the fill below el. 3800 was very nominal is important in that it may explain why the dam settled relatively greater below el. 3800 than in the lower part of Lower Bear River Dam No. 1.

Settlement During Dumping.—As dumping of rock progresses, the rock going down the slope seems to pull the previously dumped fill downward. This movement is illustrated by Figs. 1 and 4. On Aug. 12, 1929, points 1-6 were set on the top of the 3800 lift. As the fill progressed, points 7-9 were added and intermittent readings were taken. The settlement of points 6-9 is the full settlement which took place during the dumping from the 165-foot lift, except for the two weeks that the rock was from 0 to 25 feet from the edge of the berm. Point 6 settled 1.9 feet vertically plus an estimated 1.0 feet during the first two weeks for a total of about 3 feet, or about 2% for the 165-foot lift. This is perhaps more than should occur with a fill of similar rock sluiced in accordance with present practice. The rate and amount of the

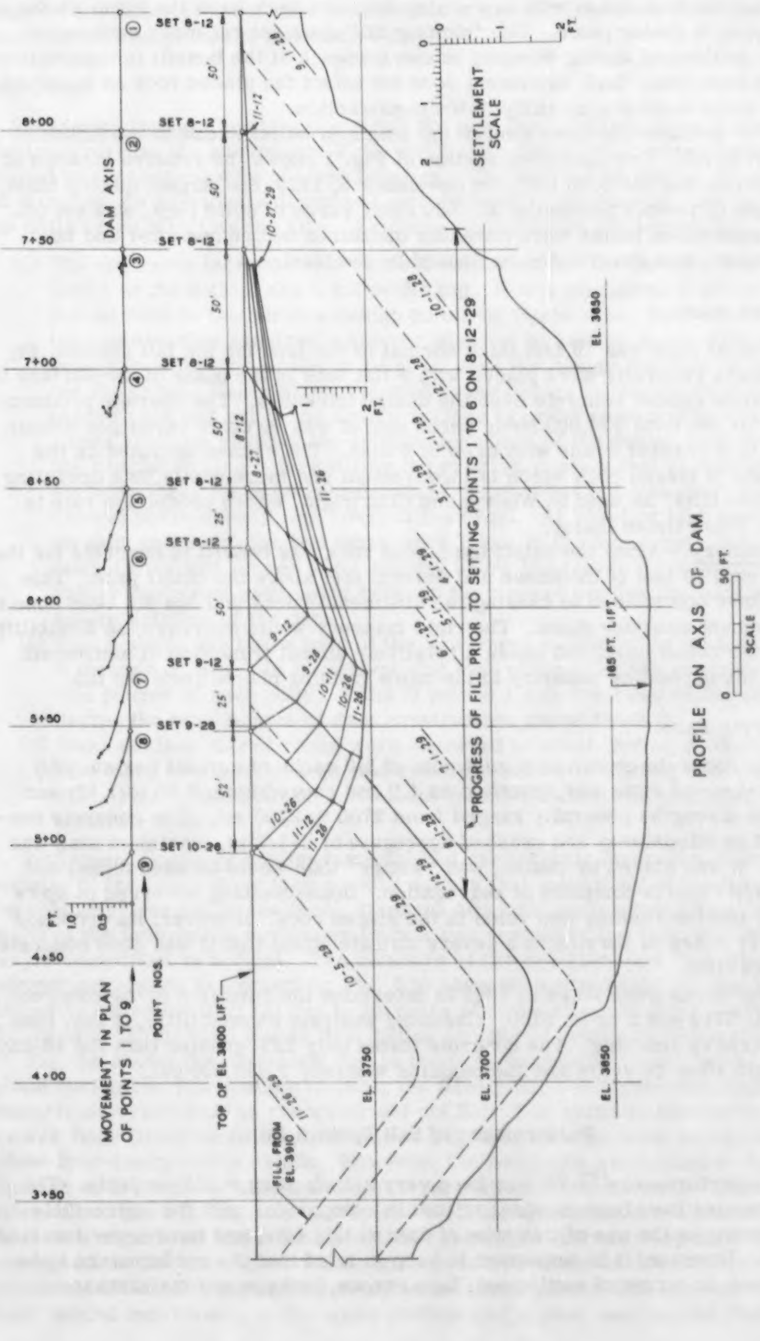


FIG. 4 - SETTLEMENT DURING DUMPING OF MAIN LIFT — SALT SPRINGS DAM

movement diminishes with increasing distance back from the berm where dumping is taking place. The "closing fill" does not get much settlement. This settlement during dumping is one evidence of the benefit in compaction from high lifts. Such movement does not affect the placed rock or concrete face since it occurs so early in the construction.

The downstream movement of the points is believed due to the influence of the 3710 lift. The maximum section of Fig. 1 shows the relative location of the points and the 3710 lift. On November 5, 1929, the largest quarry blast, 58 tons of powder producing 232,000 cubic yards of solid rock, was set off. The settlement points were carefully measured before and after and no movement was observed to be caused by the blast.

Placed Rock

Placed rock was 15 feet thick normal to the face for the full section, Fig. 1. Rocks generally were placed with a flat face in the plane of the surface to minimize excess concrete over the design thickness. The average production rate for the total 224,000 cubic yards placed was 50 cubic yards per 9-hour shift of a crawler crane with crew of 8 men. The cranes operated on the surface of placed rock which is inconvenient and more costly than operating on "face lifts" as used at Wishon and Courtright, where production rate is about 2-1/2 times faster.

Masonry.—Along the cutoff the placed rock was bedded in concrete for the upstream 10 feet of thickness and several feet above the cutoff joint. This may have contributed to causing the abutment cracks and has not been done on subsequent company dams. The rigid masonry would decrease the flexibility near the cutoff joint, and cause a relatively abrupt transition of settlement from the unyielding masonry to the more yielding placed rock and fill.

Concrete Face

The concrete contained a minimum of 5.0 sacks of cement per cu. yd.; water-cement ratio was specified as 1.0 and ranged from 0.95 to 1.10; and 28-day strengths generally ranged from 2700 to 3200 psi. The concrete contained no admixtures and crushed aggregate of 2-1/2 in. maximum size was used. It was placed by chutes; was "wetter" than would be used today; and the field reports complain of segregation. Honeycombing occurred in spots due to mortar running into voids in the placed rock. However, its condition after 27 years of service in a severe climate attest that it was good concrete for that time.

Four cores were taken in 1946 to determine the condition of the concrete, 2 at el. 3712 and 2 at el. 3780. Chemical analysis showed little, if any, loss of cement by leaching. The concrete tested only 22% greater than the 28-day strength after 16 years and the modulus was only 2,680,000 psi.

Performance of Salt Springs Dam

The performance[21,22] has been very satisfactory in all respects. The face repairs have been nominal in cost in comparison with the appreciable cost saving in the use of this type of dam at this site, and have never involved safety. However, it is important to keep in mind that the performance to be reviewed, in terms of settlement, face cracks, leakage and maintenance,

would be much better for a dam constructed at the same site with the experience and knowledge that is available today.

Settlement

Settlement During Construction.—Salt Springs Dam settled more during construction than the subsequent P. G. and E. Co. dams which were thoroughly sluiced in accordance with current practice. The settlement during the main lift dumping, as previously discussed, occurs so early in construction that it has little or no effect on the face. Evidence of additional settlement is:

(a) The upstream face was laid out to a concave slope up the face with 1.45:1 at the bottom and 1.1:1 at the top. It was attempted to place the placed rock to the surface design points on the profile. Settlement during construction required a steeper slope to be laid up to get back to the design location and finally to be at or near the design location on reaching the crest. Concrete was poured to the design thickness over the placed rock and not to the design surface in order to conserve concrete. Cross sections showing the relative locations of the design face and of the concreted face before filling the reservoir, show that at maximum sections these faces on completion of the dam were 3 feet apart in the central portion and 2 feet apart at the crest. At 150-foot high sections on each abutment, the two faces were 1 foot to 0.5 foot apart. Since the placed rock was placed to or slightly below the design lines, these values represent settlement between the time of placing the face rocks and completion of the dam.

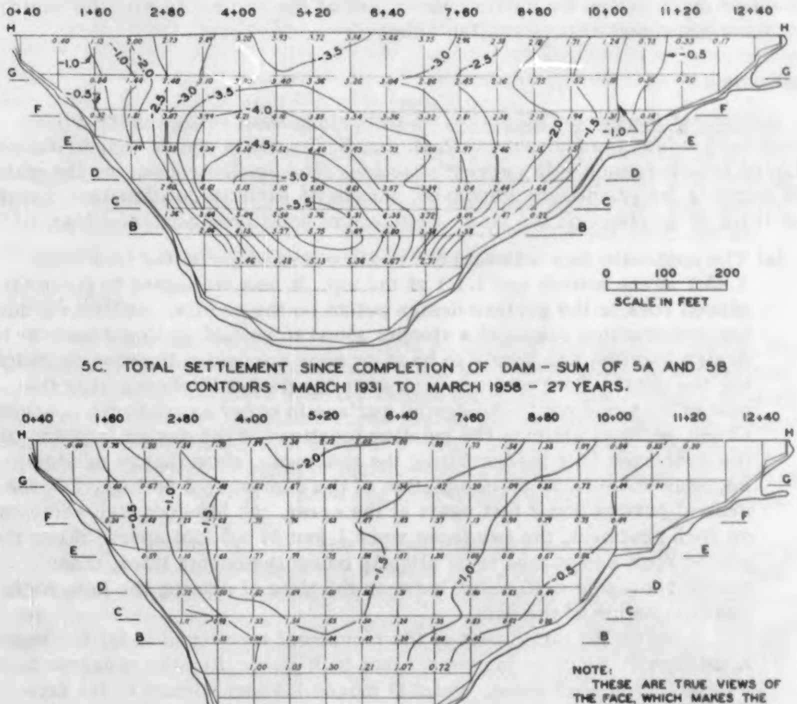
(b) Fig. 6 shows the latter part of the movement mentioned in (a) for Points A through F, which is movement that took place after the concrete face was poured at each point. Point D moved 1.4 feet normal to the face during the nine months of dam construction above Point D.

(c) Some surface placed rocks were observed to crush during construction.

(d) The many abutment cracks that appeared before the filling of the reservoir, Fig. 6, were due to settlement during construction and after the concrete face was poured.

Settlement Contours.—A fundamental and particularly favorable characteristic of the impervious face rockfill dam is that movement of the face is "essentially" normal to the face. Stresses and joint movements occur in the plane of the face due to deviations from a line normal to the face, but the major component is normal. The contours of the resultant and "essentially" normal settlement are drawn on Fig. 5 to show the settlement: (1) due to first complete water loading; (2) due to 25 years of aging after the first complete filling; and (3) the total due to first filling and the 27-year life of the dam.

Fig. 5A shows the movement due to the first complete filling. This took place during two years since in 1931, the first year, California experienced a historic dry year and the reservoir did not fill. The shape is similar to the Lower Bear River Dam No. 1 contour of Fig. 14 with maximum movement at about four-tenths of the height. However, the maximum settlement at Salt Springs is 2.7 (4.32 + 1.60) times the crest settlement whereas at Lower Bear River Dam No. 1 it was only 1.4 (2.05 + 1.45) times the crest settlement. This is a significant difference and it is considered that the very high movement in the lower part of Salt Springs is due to the inadequate sluicing below el. 3800. The high settlements in the lower part of Salt Springs caused high lateral movements in the upper portion of the dam, and caused abutment



5C. TOTAL SETTLEMENT SINCE COMPLETION OF DAM - SUM OF 5A AND 5B CONTOURS - MARCH 1931 TO MARCH 1958 - 27 YEARS.

5B. DUE TO AGING AND REAPPLICATIONS OF WATER LOAD AFTER FIRST COMPLETE FILLING - MARCH 1933 TO MARCH 1958 - 25 YEARS

5A. DUE TO FIRST COMPLETE FILLING - MARCH 1931 TO MARCH 1933 - 2 YEARS

FIG. 5 SETTLEMENT CONTOURS - SALT SPRINGS DAM

cracks in addition to those that occurred during construction. It is important to keep in mind that the large resultant settlements in the lower portion of the dam, and their unfavorable consequences of lateral movement and cracks, are excessive primarily because of the inadequate sluicing.

Fig. 5A shows that the combination of water pressure and height of rockfill that give maximum movement is located near four-tenths the height of dam. All other points may be considered to move more than would be caused by the water load at each point due to the drag from the area of maximum movement. For example, there is no water pressure at the crest which at the center moved 1.60 feet.

Fig. 5B shows that the settlement due to the aging of the fill and due to the reapplication of water load each year is different from that due to the first filling. The movement seems to be due more to the weight of the rockfill than due to the reapplication of water load. The high movement at the crest is probably due to the fact that the rock points in the upper portion were never very heavily loaded and aging is more effective on these less stabilized rock contacts. It is very favorable that the repeated reapplication of the substantial water load does not appear to cause significant movement of the rock.

Fig. 5C shows the contours of face movement for the 27 years since the beginning of the first filling, and is the sum of Contours 2A and 2B. Except in an area along the abutments, this settlement has caused no cracks. The reference plane for the contours is the position of the face on 3-9-31, just prior to the first filling of the reservoir, as shown on Fig. 3. The concrete slabs in the lower and central portion of the dam have moved even more than the contours show, as is shown by the location of points when set, Fig. 6.

Settlement at Maximum Section.—Fig. 6 shows the record of vertical and horizontal movement for the maximum section. The fact that the reservoir did not fill the first year made it possible to obtain information on the full face due to the partial filling. Some comments on Fig. 6 are:

(a) Point B moved 2.9 feet due to the water surface rising to el. 3910, and 1.0 feet due to the application of the next 38 feet of head, estimating 0.1 feet of the 1.1 feet was due to reapplication of the 166 feet of head. This indicates an increasing rate of settlement as head increases. This is equivalent to saying that the coefficient of settlement increased as head changed from 166 feet to 204 feet.

(b) Point G moved 0.1 feet as the reservoir refilled to 3910 and another 0.9 feet due to the 38 feet of additional head.

(c) Settlement is down from normal. The slabs DE, DC, CB, and BA were all in compression in a line down the face as determined by the shortening. The horizontal joints were cold joints so that this shortening caused compressive stresses. For example, the 44-foot slab length CB has shortened 0.1 foot without crushing. Slabs HG and GH are under tension in a line up the face and horizontal joints G and F have opened to relieve that tension.

(d) Vertical crest settlement. 3.6 feet of the 6-foot overbuild has been used up in the 27 years to 1958.

(e) As also shown by Figs. 5 and 7, the maximum rate of settlement after the first filling is at the crest and not in the lower portion of the dam. This is favorable, since it indicates that the crushing of rock points in the lower portion of the dam, due to the first application of water load, leaves a rock mass capable of resisting repeated application of that

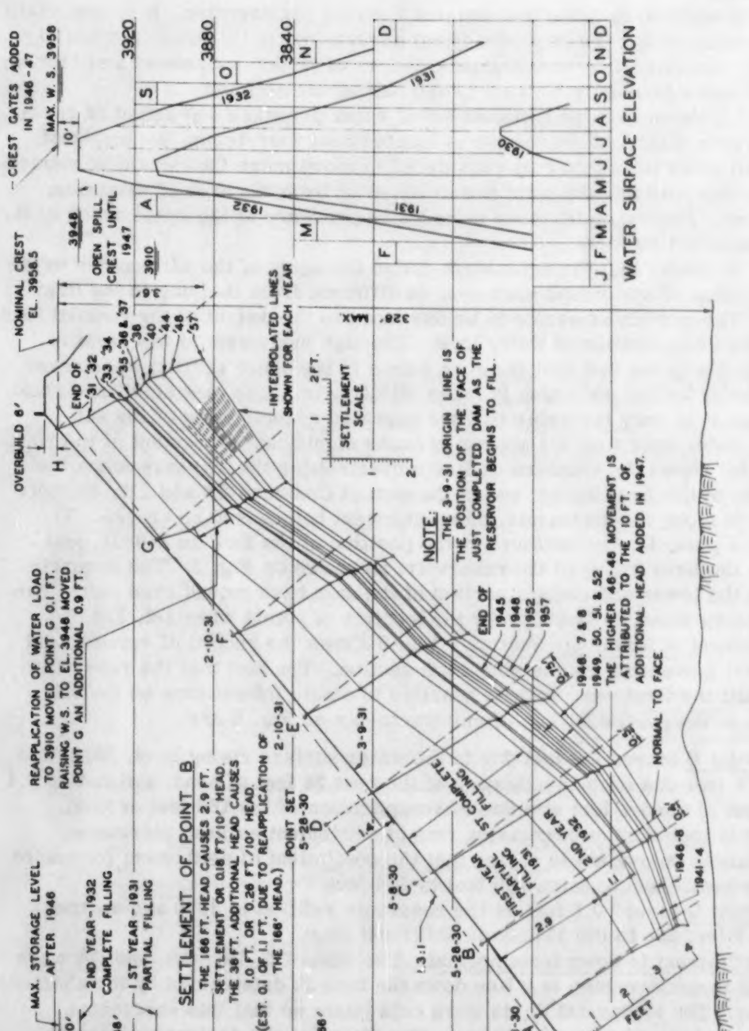


FIG. 6 SETTLEMENT AT MAXIMUM SECTION - STATION 4+60 - SALT SPRINGS DAM.

heavy load with nominal additional settlement. The rock points in the upper portion of the dam, that have not been loaded as heavily by rock or water loading, settle at a higher rate in aging than does the more heavily loaded rock points.

(f) After the first filling, the settlement is increasingly down from normal to the face.

(g) The addition of crest gates in 1946 added 10 feet of head that influenced settlement for the 3-year interval from the end of 1945 to the end of 1948. It is favorable that the 10 feet of additional head caused nominal movement. This seems inconsistent with the comments of (a) above, except that 15 years of aging settlement took place before this additional 10 feet was added to the maximum storage level.

Settlement of Points H and F.—These two points on the maximum section, Fig. 5A, are used to illustrate the first three years of settlement in detail. The settlement is plotted, Fig. 7, with movement in feet as abscissa and at the elevation of the water surface when the reading was taken. The number of months between readings is shown on the line between points. The progressive first three years of movement is summarized on Fig. 7 for the vertical settlement of Point H.

These charts show movement for the first three years in greater detail than Figs. 4 and 5 and are typical points. Both points exhibit similar characteristics. During the several months of drawdown (approximately 80 feet) after the first, second and third filling, Point H and similar adjacent points showed vertical rebound of 0.01, 0.02 feet or zero settlement. During the remaining 120 feet of drawdown, taking place during 6 or 7 months, Point H settled vertically some 0.16, 0.8, and 0.4 for the first, second and third year. The horizontal movement was quite different. Point H continued to move horizontally during the first several months of drawdown (approximately 80 feet) and then would rebound horizontally 0.03 to 0.04 feet while it was settling vertically. Horizontal rebound of Point F was 0.04, 0.09 and 0.01 feet, which is greater than vertical rebound. It appears that permanent horizontal settlement takes place while reservoir is substantially filled and a horizontal force is present. However, the vertical settlement goes on while reservoir is empty due to the weight of rock. This is one indication of the influence of reservoir operation on the pattern of settlement. The reservoir operation of 1932, as shown on Fig. 6, is typical of Salt Springs operation.

The bar graphs of settlement, Fig. 7, show the rate of settlement after the large movement due to the first complete filling. The crest, Point H, has settled vertically at about 0.09 feet per year for the twenty-five years, with slight decrease in rate. The horizontal component has decreased in twenty-five years from 0.07 to 0.025 feet per year. At Point F, located 92 feet lower than the crest, the vertical rate of movement has decreased from 0.76 to 0.44 foot per year, but has been constant for the last thirteen years at 0.44 foot per year, just one-half the rate of the crest, Point H. On the other hand, horizontal movement of Point F has been about the same as that of Point H in the last ten years, and is very small at about one-quarter inch per year.

Lateral Settlement.—The term lateral settlement refers to the horizontal movement in the plane of the face of the dam. Such movement causes opening and closing of the vertical joints and/or stresses across the face slabs. Fig. 8 presents lateral measurement readings for four significant dates: (1) just prior to first filling; (2) after first year partial filling and unwatering;

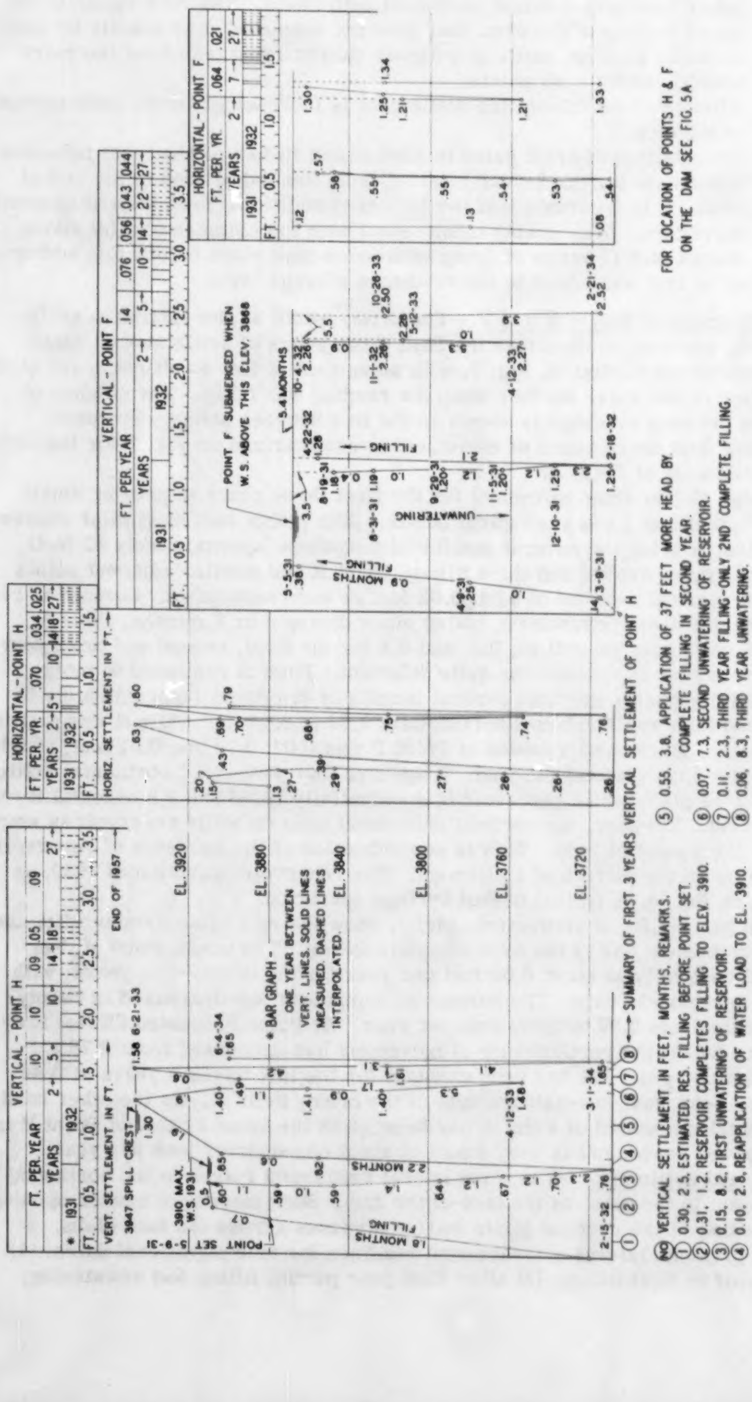


FIG. 7 SETTLEMENT OF POINTS "H" AND "F" OF MAXIMUM SECTION - STATION 4+60 - SALT SPRINGS DAM.

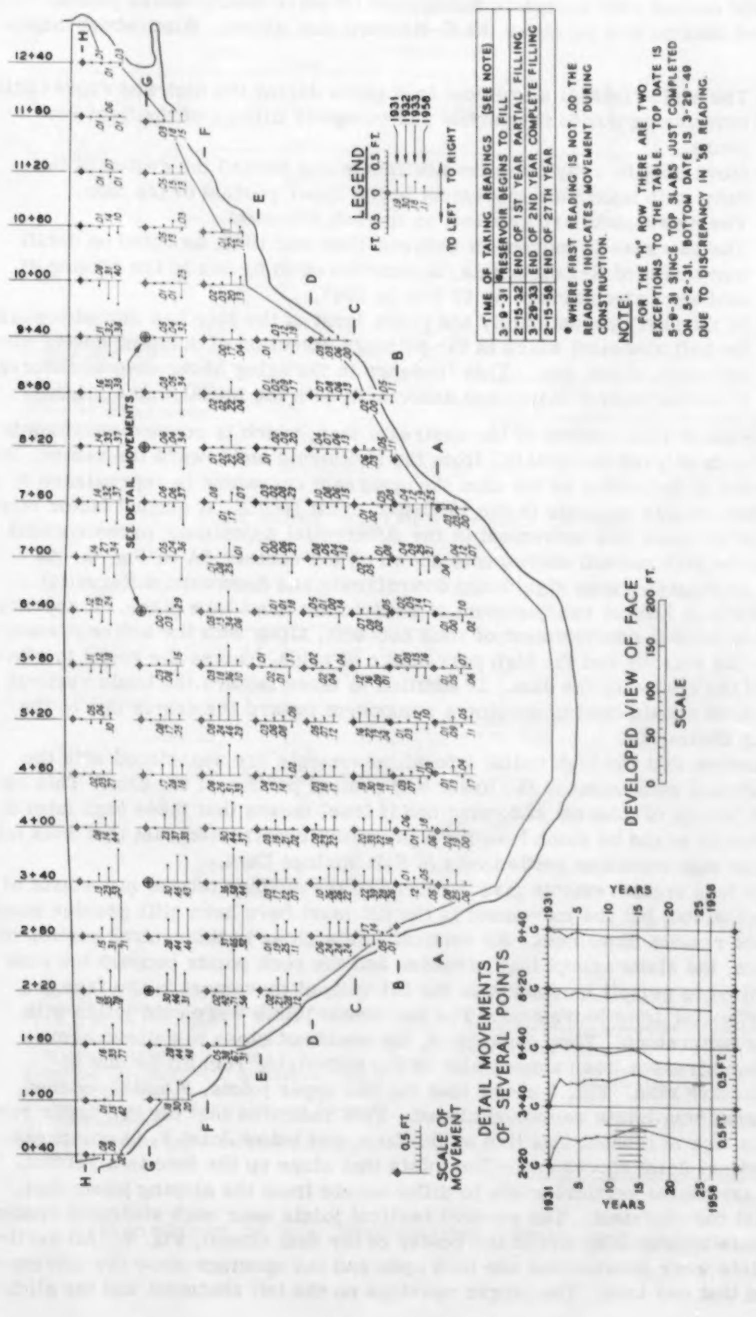


FIG. 8 — LATERAL FACE MOVEMENT — SALT SPRINGS DAM

(3) after second year complete filling; and (4) after twenty-seven years. The detailed data on five points of the G-line are also shown. Some observations are:

(a) The major lateral movement took place during the high and rapid initial movements due to the partial and complete fillings of the first two years.
(b) Movement is away from the abutments and toward the center of the dam, with maximum movement in the upper portion of the dam.
(c) The movements are greatest on the left abutment.
(d) The increase in movement between 1946 and 1949, as noted on detail movement of G-Line points, is considered to be due to the raising of storage water surface by 10 feet in 1947.
(e) In the past approximately ten years most of the face has moved toward the left abutment which is the primary direction of dumping during construction of the dam. This tendency in the aging of the dam is different from the lateral movement associated with the initial water loading.

The water load applied to the upstream face, which is convex upstream in plan, tends to push the rockfill from the abutments and toward the center. At the crest at the center of the dam the upstream curvature is represented by a 13.6-foot middle ordinate in the 1300-foot crest length. A second factor considered to cause this movement is the differential adjustment in the rockfill due to the high normal movements shown on the contour 5A of Fig. 5. As water pressure causes significant downstream and downward differential movement, a lateral readjustment of rock points must take place. It appears that this lateral readjustment of rock contacts, along with the active pressure within the rockfill and the high percentage of voids, causes the rocks to move toward the center of the dam. In addition to these factors the basic vertical settlement should tend to develop a component toward the center due to the sloping abutments.

It seems that the high initial lateral movements are associated with the high normal settlement in the lower and central portion of the dam. This apparent fact is of interest and value and if true, means that these high lateral movements would be much less for a presently constructed dam that does not have the high resultant settlements of Salt Springs Dam.

The face measurements give a record of the surface lateral movement of the face slabs, but the movement in the fill must have been still greater since the face resists movement. As vertical joints close in the central portion of the dam, the slabs accept high stresses and the rock points beneath the slab can adjust to permit movement in the fill without movement in the face.

Horizontal Joint Movement.—The horizontal joints were cold joints with copper waterstops. They could open, but could not close to relieve compression stresses due to movement of the underlying rockfill or due to temperature rise. Fig. 9 shows that the two upper joints, F and G, opened. The remaining joints remained closed. This indicates that the two upper rows of slabs are in tension in a line up the face, and below Joint F, in compression.

Vertical Joint Movement.—The joints that slope up the face in a vertical plane are called vertical joints to differentiate from the sloping joints that parallel the abutment. The several vertical joints near each abutment opened and those approaching and at the center of the dam closed, Fig. 9. All vertical joints were constructed one inch open and the openings show the movement beyond that one inch. The larger openings on the left abutment and the sliding

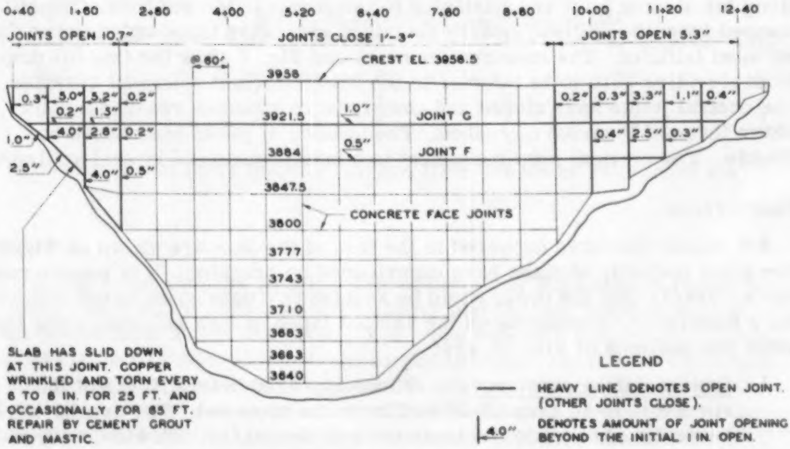


FIG. 9 FACE JOINT MOVEMENTS - SALT SPRINGS DAM

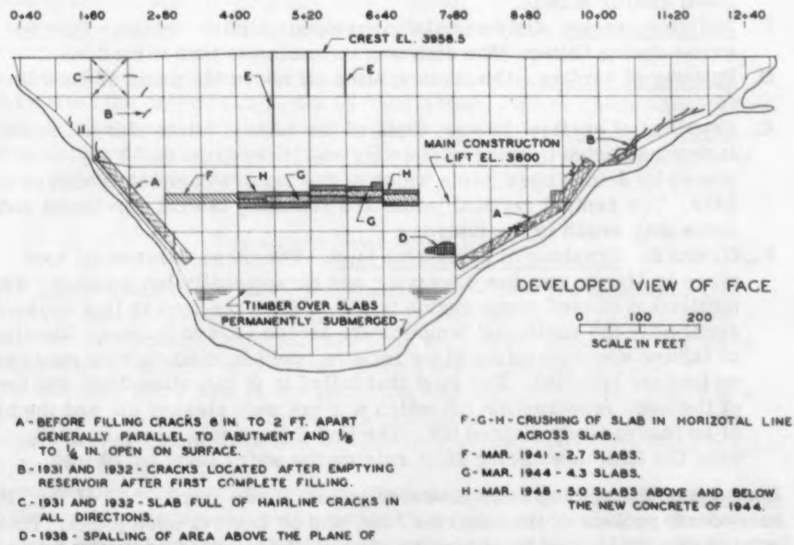


FIG. 10 RECORD OF ALL FACE CRACKS - SALT SPRINGS DAM

along the sloping joint are attributed to the construction methods. The fill dumped from the spillway quarry contained excessive fines and sluicing was not used initially. The construction notes and Fig. 2 show the thin fill dumped from elevation 3910 to be inferior to the fill from right abutment quarries. The central joints have closed and compressive stresses reached a point where three of them have crushed. The opening of joints has caused no trouble. The vertical joint movement is a manifestation of lateral settlement.

Face Cracks

All cracks that have occurred in the face of the dam are shown on Fig. 10. The great majority of slabs have experienced no cracking. It is considered that all cracks that did occur could be avoided in a dam constructed with today's knowledge. Comments on the various types of face failures, using the letter designations of Fig. 10, are:

A. Cracks during construction. These appeared before filling of the reservoir in an area 10-30 feet from the abutment. The cracks were generally parallel to the abutment and about 1/16 inch wide, with maximum being 1/2 inch. Initial repair was by chipping and using expansive mortar. With time, further joint opening and frost action resulted in deteriorated concrete zones. These areas were chipped out and replaced with new concrete, the last of such repair being made in 1958.

B. Cracks during filling. The few cracks that developed were repaired by grout and/or mastic.

C. Hairline cracks. Only one slab developed hairline cracks. This occured during filling. The slab was in tension in both directions.

D. Spalling of surface. One area spalled off above the plane of the reinforcing.

E. Crushing of vertical joints. Eight of the central joints closed the one-inch constructed opening completely and three crushed. They were replaced by 3-inch open joints which at the top are completely closed in 1958. The central vertical joints are resisting lateral movement and some may crush in the future.

F., G. and H. Crushing of horizontal joint. The three failures all took place in March with low reservoir and exceptionally hot weather. The settlement caused compression in a line down the face in this area and apparently the additional temperature stress caused failure. The time of failure was convenient since repairs could immediately be made and no leakage resulted. The joint that failed is at elevation 3800, the level of the main construction lift which was not well cleaned up, and the top of an inadequately sluiced lift. The last failure took place in 1948, upon the first unwatering after raising the water level by 10 feet.

Leakage.—Leakage has been produced primarily by cracks "A" of Fig. 10, by honeycomb pockets in the concrete face, and by several open joints. Points of leakage may be located by observing clumps of pine needles on the face of the dam. As long as leakage did not approach or exceed the required release, face repairs were not made.

A tabulation of leakage in cfs for the last 10 years from the 380,000 square foot surface and the abutments of Salt Springs Dam is:

Water Level	Depth Feet	1947	1948	1949	1950	1951	1952	1953	1954	1955	1956	1957
		Leakage At Different Reservoir Levels										
3958	320	13	20	16	19	8	12	14	16	16	11	16
3900	262	6	9	7	7	3	3	6	8	8	4	8
3850	212	2	4	3	3	2	2	3	3	3	3	3

It is observed that there is little leakage from the lower 212 feet of the face.

Maintenance

Crack and Frost Damage Repair.—Maintenance has consisted of repair to the cracks and crushed areas described above and of local small concrete patches where frost action had taken place. The concrete poured in 1931 had occasional porous or honeycomb areas on the surface. Frost action enlarges such zones and also the concrete adjacent to cracks. The patches arrest the frost action and have been satisfactory. Frost action has been worse on the right abutment side of the face which is exposed to winter sun, and more cycles of freezing and thawing. The left abutment is in the shade during the winter.

Cost.—In 27 years of life of the dam, the maintenance cost has averaged \$8,000 per year. This compared with a \$2,000,000 estimated saving, in 1931 dollars, in using a rockfill rather than a concrete dam. Maintenance has not affected operation and the cost has been nominal.

Present Condition.—A thorough inspection of the face was made in February 1958. The original concrete appears to be sound, except in small local areas affected by frost action. Nominal patching of such local spots should arrest the deterioration due to frost action. Water chemical action on the face has removed very little surface mortar and presents no problem. Nominal future patching should extend the life of the slab into the indefinite future. The dam is in excellent condition.

The Lower Bear River Dams

The Lower Bear River Dams No. 1 and No. 2[23,28] are located on the Bear River just below the Upper Bear River rockfill dam, are only a few miles from Salt Springs Dam, and are near the Carson Pass Highway about 40 miles out of Jackson, California. They were completed in 1952.

The 245-foot No. 1 dam is the main dam and the 150-foot No. 2 dam is a wing dam, the two being separated by a granite knoll, Fig. 11 and 19. The reservoir stores 49,000 acre feet for use in a power drop of one mile in four P. G. and E. Company plants.

In connection with the Lower Bear River dams, the purpose of this paper is to thoroughly present the performance data and more briefly present design and construction data. Comparisons will be made with Salt Springs Dam. The two Lower Bear River Dams will be discussed separately.

Design of Lower Bear River Dam No. 1

The site was exposed massive granite requiring no streambed excavation and nominal excavation of loose material from pockets on the abutments. The

concrete face rockfill dam was selected on the basis of lowest estimated cost and adaptability to the site and to the operation of the reservoir.

The outlet works are illustrated and described in references (23) and (28). The spillway is a double side channel spillway located on the knoll between the two dams and discharges in an unlined channel, Fig. 11. The design capacity is 12,000 cfs. This discharge is passed with a depth of 4.6 feet over the open crest weir, and 3.4 feet of freeboard on the 4.0-foot high coping wall.

The main design features are as shown on the section, Fig. 11, and the joint arrangement is as shown on Fig. 14. The face is moderately curved in plan, and is the shape of a cylinder of 4350-foot radius tilted on a 1.3:1 slope. At the crest the center arches 14 feet upstream and at 0.4 height only 6 feet. This arching is just enough to give a favorable appearance after settlement and to give a tendency toward compression across the face rather than tension. The moderate compressive stress and closing of joints is preferred.

The 1.3:1 upstream face slope requires greater quantities than the 1.33:1 to 1.1:1 slope for the top 245 feet of Salt Springs Dam. The exposed rock foundation and the narrow canyon made the greater quantities nominal and it was thought that lower bid prices due to convenience of building the face to the natural slope would offset the cost associated with the increased quantities. Observations during construction indicated that this was not the case, and the subsequently designed Wishon and Courtright face slopes are variable and are steeper than that of Salt Springs.

The Lower Bear River designs are significant as a transition between the early Salt Springs design and the recent Wishon and Courtright dam designs. The remaining significant features will be reviewed in comparison with Salt Springs.

Comparison with Salt Springs Design

Improvements.—Though the face troubles at Salt Springs were not serious, it was desired to obtain a more watertight dam at Lower Bear River. Also, the minimum reservoir operating level is 120 feet above the base of the dam, and unwatering the reservoir would require a shutdown of the plant and a loss of 2200 acre feet of dead storage. With the knowledge gained from Salt Springs, it was considered that a more watertight face could be obtained. Improvements to minimize settlement, face cracks and leakage are considered to be:

(1) Sluicing: High pressure sluicing, with water to rock ratio of 3 to 1 by volume, was specified to provide a fill that would give minimum settlement.
(2) Hinge Slabs: The 15-foot slabs along the abutment were provided to minimize or prevent cracks in the slabs near the abutments.
(3) Soft Horizontal Joints: To release compression that might develop in a line down the face, the horizontal joints contained one-half inch thick redwood boards in addition to the U-shaped copper waterstop, Fig. 17B.
(4) Lift Surface Cleanup: The lift surface was specified to be scarified, sluiced or otherwise prepared to obtain a rough exposed rock surface for contact with the succeeding lift.
(5) Two-Inch Open Joints: To prevent the crushing of vertical joints near the top and central portion of the dam, joints in that area were constructed 2 inches open, Fig. 17.
(6) Additional Horizontal Joints: The horizontal joint spacing is the same

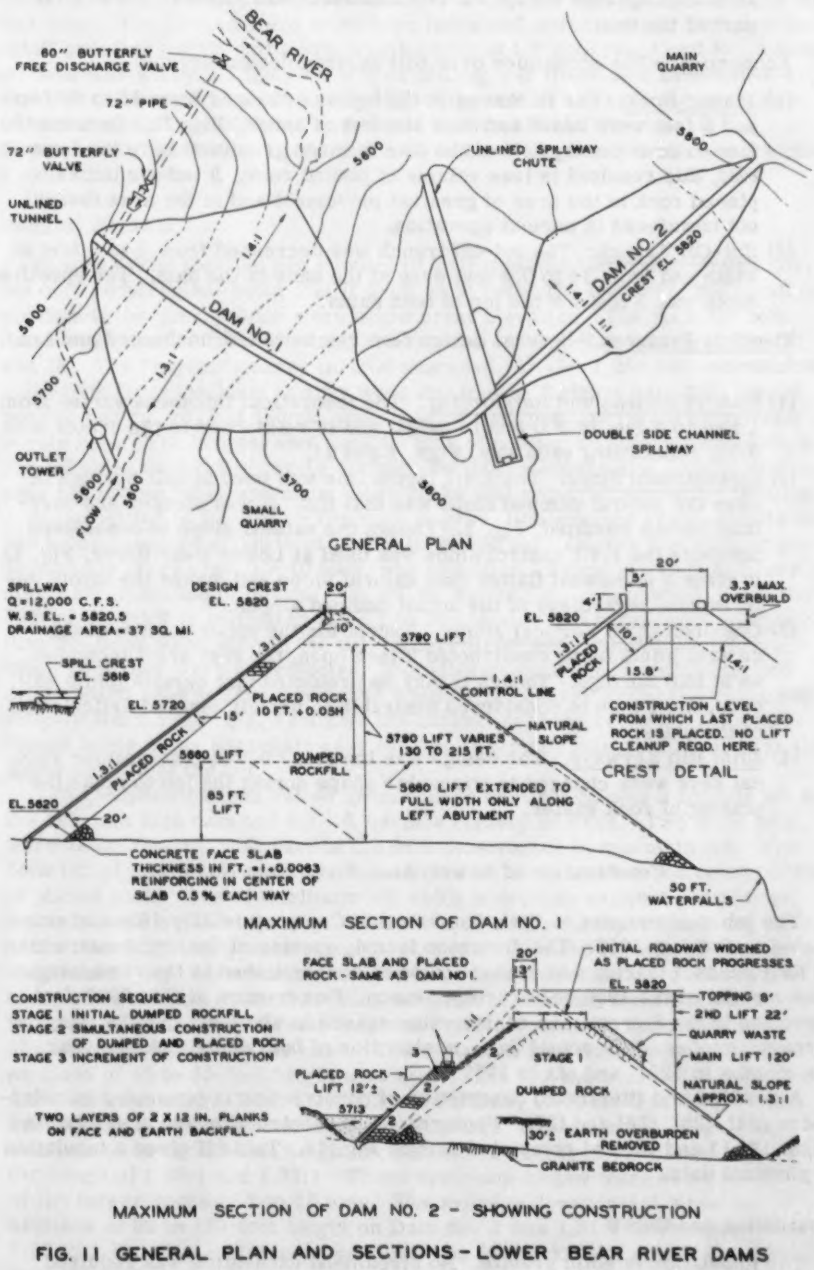


FIG. 11 GENERAL PLAN AND SECTIONS - LOWER BEAR RIVER DAMS

as at Salt Springs except for two additional soft joints in the central part of the dam.

Economies.—The economies over Salt Springs design were:

(1) Placed Rock: The thickness at the top was changed from 15 to 10 feet and 5 feet were added for each 100 feet of height, Fig. 11. Because the face area of the top half of the dam is much greater than in the lower half, this resulted in less volume of placed rock. It put the thicker placed rock in the area of greatest movement and in the area that is not unwatered in normal operation.
(2) Cut-Off Trench: The cut-off trench was decreased from 5 to 3 feet in width and from 15 to 7.5 feet deep at the base of the dam. The specified depth was 3 feet for the top of both dams.

Identical Features.—Several design features were left unchanged and are as follows:

(1) Slab Thickness and Reinforcing. The theoretical thickness varies from 1 foot to 3 feet in 300 feet of height and is reinforced in the center with 0.5% reinforcing each way, Figs. 1 and 11.
(2) Downstream Slope. The 1.4:1 layout line was used at Salt Springs in case the natural dumped slope was that flat. It was steeper and overbuilt berms resulted, Fig. 1. Though the natural slope is considered adequate the 1.4:1 control slope was used at Lower Bear River, Fig. 11. It gives a somewhat flatter than natural slope and makes the layout determinate regardless of the actual dumped slopes.
(3) One-Inch Open Vertical Joints. Except for the upper portion of the central joints being constructed 2 inch open, the rest are 1 inch open as at Salt Springs. Though it may be predicted that certain joints will open, the 1 inch is considered desirable to permit assured articulation of the slabs.
(4) Joint Rib Keyways. The design was the same but the rectangular vertical keys were changed to triangular shape during the job to make the placing of rock easier.

Construction of Lower Bear River Dam No. 1

The job was awarded to Utah Construction Company in July 1950 and completed in October 1952. The diversion tunnel, opening of quarry, construction of haul roads, clearing and excavation were accomplished in the remaining four months of the 1950 construction season. Due to snow at this 6000-foot elevation in the Sierras, the construction season is about six months, May through October. The actual dam construction of both dams took one year, six months in 1951, and six in 1952.

A detailed and illustrated description of construction is presented in references (24), (25), (26) and (27). Photographs of construction are presented in Figs. 12, 13 and 18, and completed dam in Fig. 19. Table II gives a tabulation of physical data.

Foundation and Cutoff

The foundation is solid granite. No streambed excavation was required and pockets of overburden on the abutments were removed by sluicing.

Grouting was in 25-foot deep holes at 10-foot centers with every fifth hole 50 feet deep. The 50-foot holes were core holes and water tested. Data for the cutoff grouting for the 2510-foot length of cutoff for both No. 1 and No. 2 dams is: one hole every 9.7 feet; 4.3 feet of drilling per lineal foot of cutoff; 3.4 sacks of cement per foot of cutoff; 85% of cement in holes taking more than 50 sacks; and 41 of 258 holes took more than 2 sacks of cement. In the distance between Station 1 + 80 to 6 + 80 on Dam No. 1 no hole took more than 1 sack of cement.

Dumped Rockfill

Quarry.—The location and number of quarries was left to the Contractor, the only requirement being that it must be at least 150 feet from the cutoff of the dam if the quarry floor were below crest elevation. The rock for both dams was obtained from one main quarry and one small toe quarry, Fig. 11 and 18. The "coyote" quarry method was used.(24,25) 1,260,000 cu. yds. of solid rock from the Main Quarry were obtained in 9 blasts with 0.60 lbs. of powder per cu. yd. and 460 cu. yds. of solid rock per foot of 3.5 by 5.0-foot coyote hole. The largest shot used 89 tons of powder and yielded 307,000 cu. yds. of solid rock. The coyote method produced a substantial amount of fines near the coyote holes and many large rocks that required secondary drilling.

Composition of Rockfill.—The rock is gray, fine-grained grano-diorite that has a compressive strength of 15,000 to 20,000 psi. Quarry run rock of predominantly large size, 1 - 10 tons, was specified. Fines of 4 inch and less in size were specified to be less than 5% by weight. As constructed, more than 50% of the rock is estimated to be 1 to 20-ton size and many loads with more than 5% fines were used in the fill. Where fines were dumped over large clean rock they were not rejected. Including overburden, a weathered and decomposed zone and the cleanup of fines on the quarry floor, the waste was roughly about 15%. Fig. 12 shows the character of the rockfill. Note that though large rocks segregate near the toe, there are many that remain in the upper portion of the lift.

Lifts.—Specifications called for not more than 3 lifts plus a topping lift for the 245-foot high dam and for lift surface cleanup of fines. Two main lifts were used, Fig. 11, with part of the dam constructed in one main lift. The 5660 lift of minimum top width, was to give an early start to the construction of placed rock. It is of minimum top width to provide earliest completion, and minimum surface cleanup. Also, it is of minimum top width to let some of the next lift be of greater height. However, in order to keep all shovels and trucks busy, a portion of the 5660 lift was extended to the downstream slope along the left abutment. Fine material cleaned up from lift surfaces was cast over the downstream face and gives the dam the appearance of being of smaller rock than is the case. The upstream face dumped slope on nine sections of 45 to 85-foot height averaged 1.28:1 and the horizontal distances for 1 vertical were: 1.25, 1.27, 1.21, 1.27, 1.26, 1.33, 1.34, 1.33, 1.24. A dumped slope of large rock is quite irregular. The upstream slope of 20 sections of 40 to 105-foot height on Dam No. 2 averaged 1.24:1 and were in the range of 1.20:1 and 1.32:1. These upstream slopes were of selected loads of the larger rocks of 2 to 15 tons. The average downstream slope for 16 sections of 60 to 120-foot height on Dam No. 2 was 1.31:1 and ranged from 1.26 to 1.40. The 1.31:1 average slope was for rockfill of smaller and more broadly graded rock than the upstream slopes.

FIG. 12 DUMPING ROCK ON 120 FT. LIFT
LOWER BEAR RIVER DAM NO 2

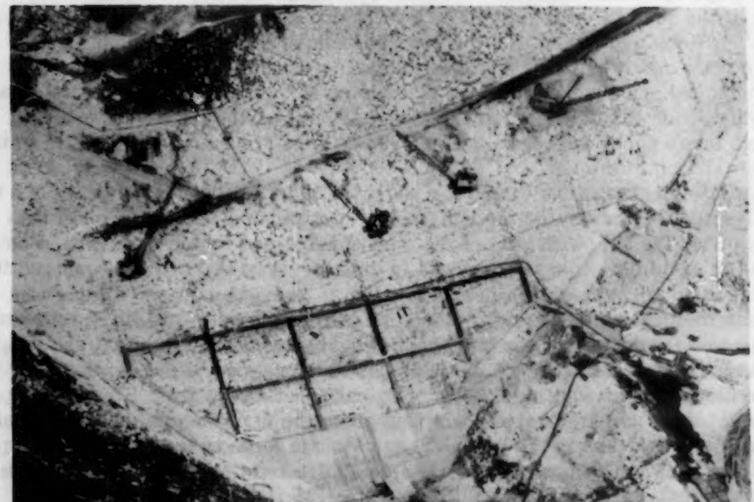


FIG. 13 UPSTREAM FACE SHOWING DUMPED ROCK, PLACED ROCK AND CONCRETE FACE CONSTRUCTION
LOWER BEAR RIVER DAM NO. 1

Sluicing.—Sluicing was at a volume ratio of water to rock of 3:1. The ratio is rather arbitrary and if the rock is relatively clean 2:1 would probably be quite adequate. A cost study showed that for the 2:1 ratio the cost was 4.6 cents per cu. yd. and 75% of the cost was the labor cost of the monitor operator. For 3:1 the labor is the same and the increased pumping energy and piping cost only increased the unit cost by 0.4 cents. Therefore, the 3:1 ratio was adopted.

Four monitors were used with 2-1/2 or 3-inch nozzles and 80 to 90 psi pressure at the nozzle. A pumping plant of 3 electric 10 x 12-4210 gpm-275-foot head Ingersoll Rand pumps were used. Normal operation was with 2 pumps and 3 monitors. Piping began with a 12-inch line and branches to 8 or 6 inch lines with 25-foot lengths of 6 or 8-inch heavy rubber hose leading to the monitors. Monitor discharge ranged from 2 to 4 cfs. At Salt Springs, with no pressure, the sluicing water softened or moved dirt and quarry dust but did not excavate pockets of spalls as did the jet sluicing at Lower Bear River.

Settlement During Construction.—During the 6 months of 1951 construction season 70% of the dumped rock and 14% of the placed rock was constructed on Dam No. 1. The end of the first 3 months of the 1952 season saw the dumped rock nearly finished and the placed rock just reaching its peak rate of 15,000 cu. yds. per month. Except for several slabs poured at the base of the dam in 1951 to serve as the diversion dam, all slabs were poured in late 1952. The placed rock was placed to the theoretical shape of the face and significant movement after placing was not observed. The concrete was poured to the theoretical points on the face. If movement of placed rock had taken place it was intended to pour concrete to required thickness and not to theoretical layout points.

On Lines T. U and V settlement points were set on October 8, 1952 while upper slabs were being poured. On November 28, 1.7 months later, readings were taken as the dam was being snowed in for the winter. The maximum settlement for the 1.7 month period was 0.095 on the U-line at Station 3 + 80, Fig. 16.

The small movements during construction are in sharp contrast with the greater movements of Salt Springs, and the difference in sluicing method is considered to be the reason.

Placed Rock

The placed rock was specified to be of predominantly large size rock and to obtain maximum rock to rock contact with voids kept to a minimum. Voids were chinked. Rocks were set down to give forward and lateral contact as well as bottom contact. Keyways were formed in the placed rock for the joint ribs, in the same way as at Salt Springs.[22]

The placing of rocks[24,25] was from crawler cranes of 1-1/2 to 3-1/2 cu. yd. size, and as many as 7 cranes were working on Dam No. 1 at one time. The cranes operated from the top of the placed rock, Fig. 13, and moved across the dam placing a layer of 10 to 12-foot height. This required a reasonably level surface which slows down the work and was still sufficiently rough that timber pads were often used to walk the cranes. Selected loads of large clean rock were dumped on the upstream face to provide a supply of rock to the cranes, such that removal of the large rocks would not leave an inferior rockfill surface of fines and small rock adjacent to the placed rock. Handling was by cable slings and required about 6 men per crane.

The 112,000 cu. yds. required for both dams took 2980 crane shifts which gives an average of 38 cu. yds. per 8-hour crane shift. At the peak of the job 29,000 cu. yds. were placed in August, 1952 at rates of 42 and 47 cu. yds. per crane shift for Dams No. 1 and No. 2 respectively. The 38 cu. yds. per crane shift compares with 50 for Salt Springs, 100 for Bowman and 50 to 100 for some other dams. Much consideration was given to the Specifications and procedures for the Company's recent Wishon and Courtright Dams, and rates there average about 135 and peak at 200.

Concrete Face.—Concrete was specified to be 3000 psi concrete and to contain 5 to 6% entrained air. The concrete face(27) was poured largely by Pumpcrete, but also by crane and bucket where crane access was possible. Specially designed steel forms of 2'-6" by 7'-0" dimension were continually moved ahead of the concrete on channel whalers that were fastened to the placed rock at 7 ft. spacing. The forms were stripped while the concrete was still green and the surface given a wood float. Initially, the steel forms were left in place until needed and the surface was observed to contain many air bubbles. The procedure was changed to remove them promptly and give the surface a wood float finish.. A hard dense surface is desirable to resist the frost action. The concrete work progressed very rapidly, all but the four bottom slabs being poured in August, September and early October of 1952. Due to the surface steps and irregularities of the placed rock, the average slab thickness exceeds the design thickness. This excess thickness averaged 16 inches for the dam.

Performance of Lower Bear River Dam No. 1

The service record of this dam has been very satisfactory to date, and covers a period of five years. Only two face cracks have developed, maximum leakage has been 3 cfs, and settlement has been low.

Settlement

During Construction.—Based on Salt Springs experience, it was expected that the placed rock face would settle as the dam was being constructed and that, in steepening the slope to place rock to the design lines, the shape up the face would be concave. The concrete was planned to be poured to design thickness and not to layout lines in order to avoid excess concrete and obtain the concave line. The movement was so nominal that the concrete face was poured to the design layout lines on a 1.3:1 slope. The numbers at the points on the U and V lines, Fig. 16, give the vertical settlement for 51 days between setting the bronze pins and completion of the dam on November 28, 1952 (40 days for T line). The maximum movement was 0.095 foot or 1.15 inches, which is very small.

Settlement Contours.—The reservoir is normally not drawn down below el. 5700, Fig. 15, but it was completely emptied in November 1954 which made it possible to obtain complete settlement measurements for the two-year period. These are shown as contours and as horizontal sections in Fig. 14 and as vertical sections in Fig. 16. The face has adjusted to the shape shown by the contours without cracking. Some comparisons with Salt Springs have been discussed.

It is interesting that the maximum settlement is toward the left and steeper abutment of both dams. Assuming a uniformly compacted rockfill, the point

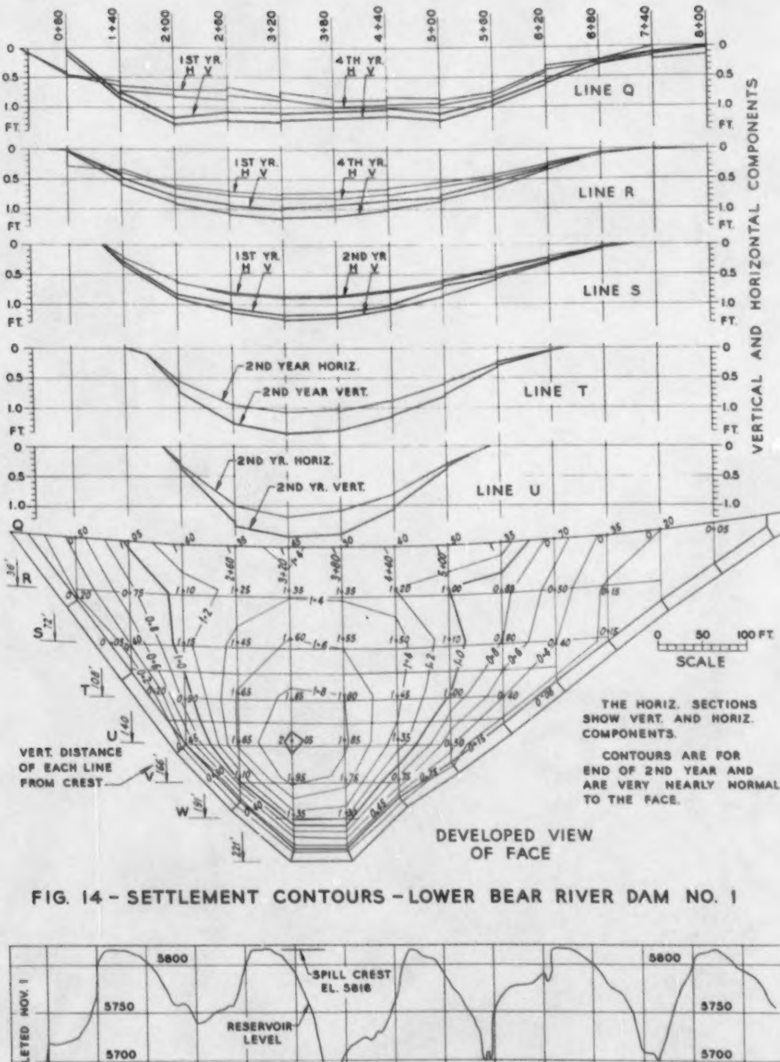


FIG. 14 - SETTLEMENT CONTOURS - LOWER BEAR RIVER DAM NO. 1

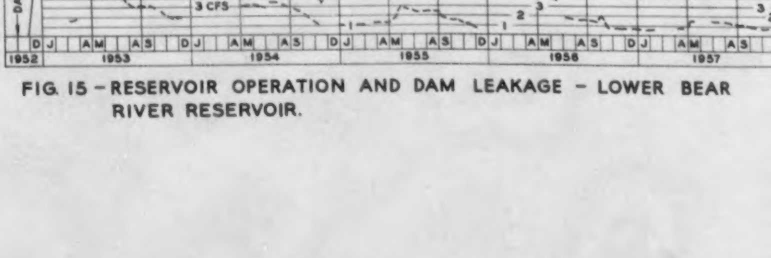


FIG. 15 - RESERVOIR OPERATION AND DAM LEAKAGE - LOWER BEAR RIVER RESERVOIR.

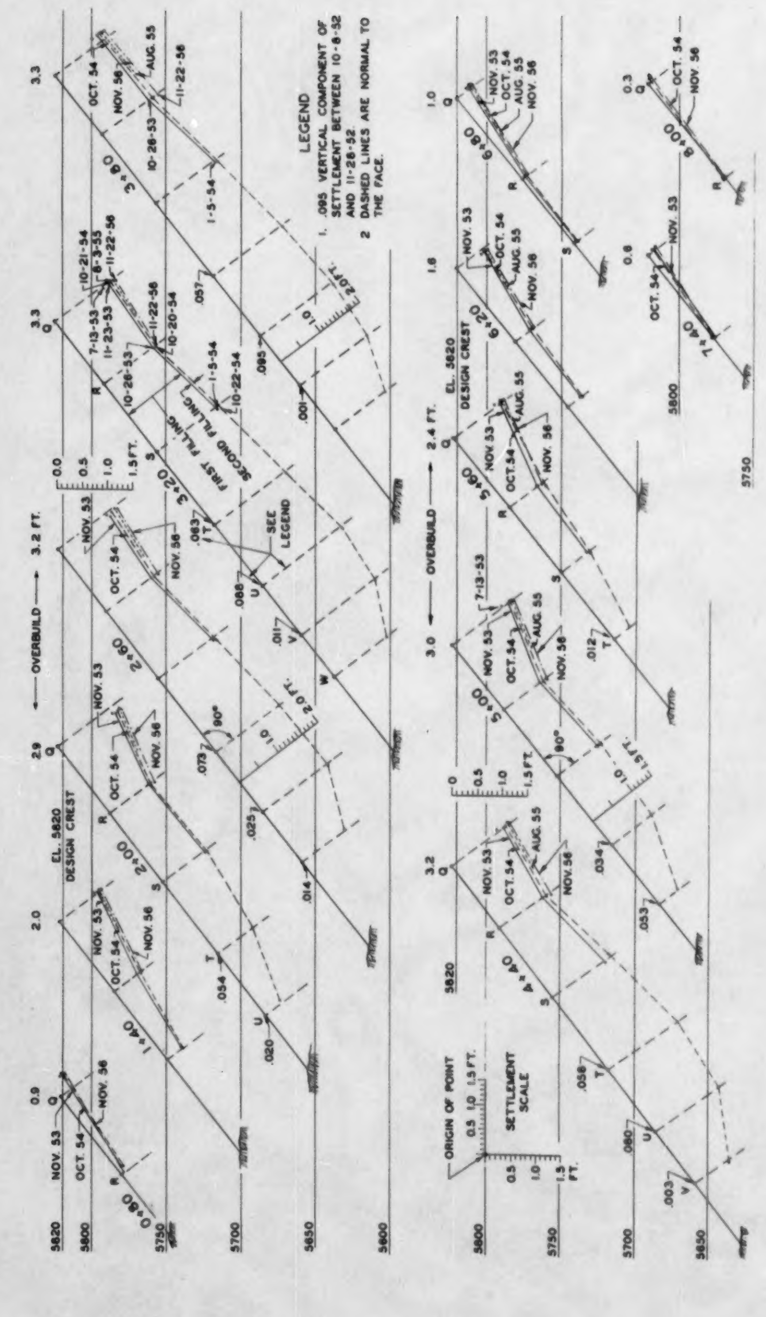


FIG. 16 SECTIONS OF DAM SHOWING SETTLEMENT - LOWER BEAR RIVER DAM NO. 1

of maximum settlement should be toward the flatter abutment. The steep abutment should resist the drag of settlement, from the high movements of the maximum section, more than the flat abutment. The tendency to higher settlements near the left abutment of Dam No. 1 is considered to be due to the construction of all lifts from right to left abutment. At Salt Springs it is considered to be due to the main lifts going from right to left abutment and also due to the inferior fill from the left abutment spillway quarry. Of course the maximum differential settlements are near the steep abutment.

Settlement Sections.—The settlement sections, Fig. 16, and Table III, show the progressive movement during four years. On July 13, 1953, six weeks after reservoir filled for the first time, crest readings were taken, Fig. 16. Five months later, after reservoir had lowered, Fig. 15, readings were again taken. The major settlement took place during the first filling. The movement during the last five months of the first year was not very great. That the face takes its major movement the first year and then moves very little is strikingly apparent and is very favorable in this type of dam. It means that if cracks are to occur, they should occur in the first year and after being repaired the dam might require little or no further scheduled unwatering or maintenance. That the movement is very close to normal to the face is also favorable since movement other than normal to the face causes stress in the slab and joint opening or closing. That the movement is essentially normal does not mean that the fill settles in the same way since the reinforced slab, that is interlocked to the large surface placed rocks, certainly resists any movement other than normal. Some readjustment of rock contacts in a zone underlying the face slab can permit the fill to move other than normal to the face without moving the face slab.

The significant settlement is that of the maximum section since its movement appears to drag the adjacent sections. This influence is seen by comparing the sections and tabulations of about equal height of Dam No. 1, Fig. 16 and Table III, and Dam No. 2, Fig. 21. Comparison of sections of equal height on the same dam further shows the influence of maximum section on the abutments of different slopes. It is considered that expressing the vertical crest settlement or the horizontal crest settlement; i.e., deflection, as a per cent of height is only significant at the maximum section if the abutments are steep. Study of the tabulations of Table III and 21 show this.

The matter of dragging from the point of maximum movement also occurs within the section. The maximum settlement due to the first complete filling is at about 0.4 height. At the top there is no water load and for the 1.3:1 face dam the water load for the lower two-thirds of the face is transmitted to bedrock upstream from the axis, and does not directly affect the rock under the crest. The movement of the crest caused by water load is due to a drag and readjustment of the rocks and it appears reasonable that maximum settlement is not at the crest.

Vertical Joint Movement.—The vertical joint movements of Dam No. 1, Fig. 17A, are markedly less than those of Salt Springs, Fig. 9. The maximum joint closure is 5/8 inch at R 3 + 20 and V 3 + 80 where the joints were constructed 2 inch and 1 inch open, Fig. 17C. All joints except 0 + 80 close slightly. The vertical joint movement is very satisfactory and confirms the design practice of leaving all vertical joints a minimum of 1 inch open. Based on the three readings on the R-Line, the joint movement took place the first year.

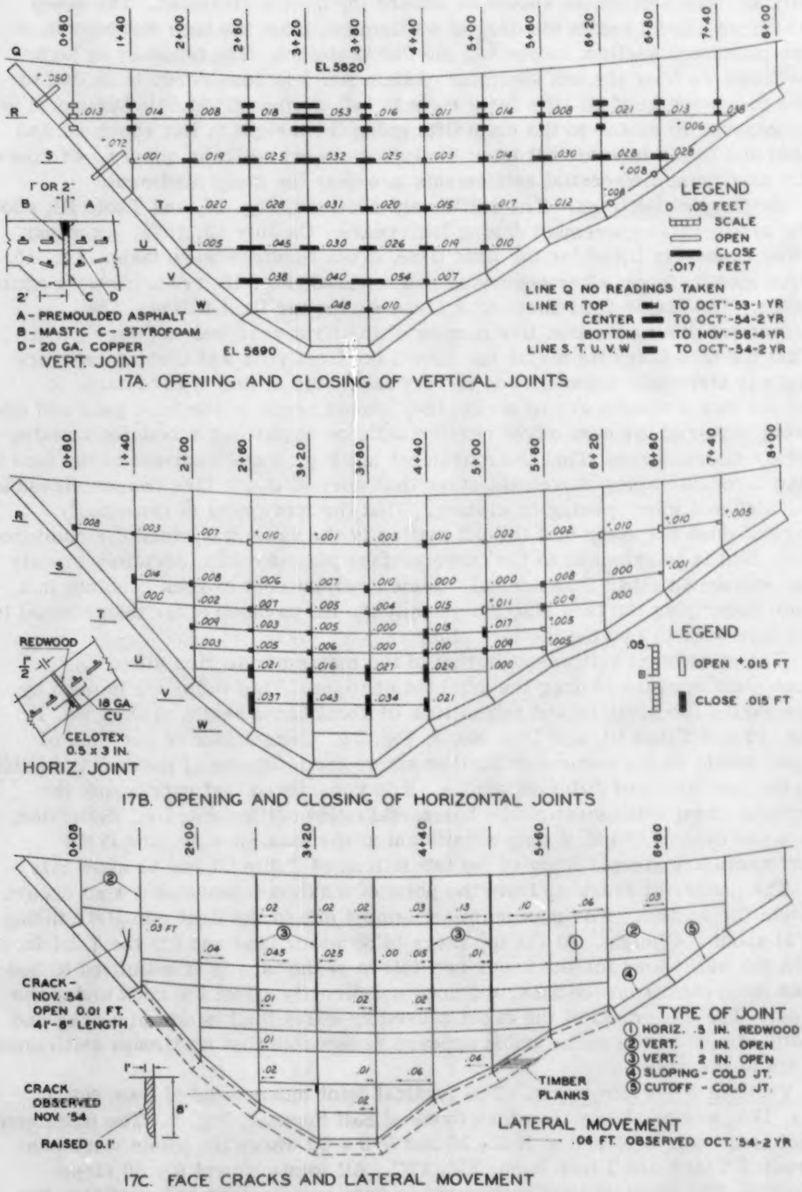


FIG. 17 MOVEMENT OF JOINTS, LATERAL MOVEMENT, AND FACE CRACKS - LOWER BEAR RIVER DAM NO. 1

Most of the joint closing is due to the resultant settlement which shortens the moderately arched face. Comparison of the net joint closing along lines R through W of Fig. 17A with the computed shortening is:

	R	S	T	U	V	W	
Close	.17	.15	.14	.14	.14	.07	feet
Shorten	.13	.13	.11	.08	.03	.01	feet

The difference is a measure of the net change in length of the total length of slabs due to horizontal stresses in the slabs. Because the central slabs are in compression and the abutments slabs in tension, the net change in length is not very great.

Horizontal Joint Movement.—Both vertical and horizontal joint movements are accurately determined by measuring between center punched 5/8-inch bronze pins on each side of the joint and near each corner of the slabs. The horizontal joints contain a 1/2-inch board of redwood and are termed soft joints. The wood filler rather than an extrudable filler was used to give some resistance to movements before yielding. Tests indicated that the redwood resisted 1000 psi when crushed to one-half thickness and 5000 psi when compressed to three-tenths its original thickness. The joint closings are all within an acceptable 0.03 feet except for several on the V-Line, Fig. 17B. The 0.043 closing at V 3 + 20 is associated with a local failure.

The top joint, R-Line, either closes very moderately or opens; all movements being less than one-eighth inch. The other horizontal joints close an increasing amount toward the bottom of the dam. Inspection of the contours and sections showing settlement make this seem reasonable. The upper portion of the face is supported by a substantial amount of rock that receives little water loading. The rock under the central and lower portion of the concrete face is pushed down without the restraint of the rock downstream from the axis and not loaded by the water. This tends to give greater compression toward the base of the dam and, in fact, tension in the top slabs.

Lateral Face Movement.—The lateral positions of points are determined by stationing along each line of points. The movements shown on Fig. 17C are in the range of 1/8 to 1/2 inch and are on the order of the accuracy of the chaining across the face of the dam. It is believed that the readings are not significant except to indicate that lateral movements are very small and that the lateral movements of the well sluiced Dam No. 1 rockfill are much less than for Salt Springs. It is repeated that the rockfill lateral movement is probably greater than measured on the slab due to the resistance of the slab and joints to movement. Measurements of the change in width of the 60-foot wide slabs show all abutment slabs to be in tension horizontally with elongations of 0.06 to 0.01 feet in 60 feet. Central slabs are in compression, having shortened generally 0.005 to 0.015. A shortening of 0.01 feet represents a compressive stress of about 700 psi.

Face Cracks and Maintenance.—Two cracks of different causes have occurred, Fig. 17C, in the five years of service. As shown on Fig. 17C, the abutment slabs, below the minimum operating level of 5700, are covered with two layers of 2 x 12-inch planks anchored to the concrete. Whether there are cracks in these hinge slabs is not known. The vertical crack near Station 1 + 40 is in a slab that has increased in width by about 0.04 feet and that slab is under tension in a horizontal line. It was filled and covered with rubberized asphalt.

The failure at V 3 + 20, Fig. 17C, is at a joint intersection where joints have closed 0.040 and 0.043, Fig. 17A and B. A 1 by 8-foot piece of concrete cracked and raised 0.10 feet. The 0.043 closure is of a joint that contains a 0.042 thickness of redwood. In order for a joint to close, the underlying concrete rib and some rock contacts must crush. With the high compression in both directions, the crushed rib could exert a large force to raise the edge of the slab. The broken zone was not removed to see what happened, but plastered with rubberized asphalt.

Leakage.—A V-notch leakage weir is located on massive exposed granite in the streambed below the dam. The leakage has never exceeded the required release and the maximum in 1957 was less than 2 cfs, Fig. 15.

Lower Bear River Dam No. 2

The No. 2 dam is a relatively small 150-foot high wing dam to the Main No. 1 dam, Fig. 11, 18 and 19. It is similar to Dam No. 1, but being of only 150-foot height some economies and simplifications in design were made. The No. 2 dam will be discussed in relation to Lower Bear River Dam No. 1.

Design

Features the Same as Dam No. 1.—The overall specifications were the same. The placed rock thickness, crest width, face reinforcing and cutoff design were alike. The vertical joints were the same, except that none were 2 inch open. The 1 inch joint was considered more than necessary for this low dam with gradual profile but was used as a practical minimum.

Features Different From Dam No. 1.—The primary difference in design is the upstream face slope of 1:1 as compared to 1.3:1 for Dam No. 1. For Dam No. 2, drill holes indicated that considerable earth overburden would have to be removed and minimum base thickness of dam was desirable. The 1:1 slope was adopted as being economic and suitable for the entire 150 feet of height of Dam No. 2. It was believed that the 1:1 face would be more difficult to construct. Observation of the simultaneous construction of the two dams of different face slopes indicated that unit costs should be about the same and placed rock was actually placed at a faster rate on Dam No. 2. Neither the "soft" horizontal joint nor the hinge slabs were considered necessary for the low height and gradual profile of this dam, and they were not used. Fewer horizontal joints were used and the crest alignment was straight.

Horizontal Joints.—The horizontal joints are essentially construction cold joints with copper waterstop and the reinforcing does not pass through the joints. At elevation 5713 the joint is useful in providing articulation of the face slab and also as a construction joint. The slabs below el. 4713 were covered with two layers of 2 x 12-inch planks which were used as forms and the hole below el. 5713 was then backfilled with earth, Fig. 11 and 20. In addition to providing a water seal the backfill gave construction road access for placing rock and concrete. The 5770 joint was for construction reasons.

Construction

Dumped Fill.—Dam No. 2 was constructed essentially in one lift, which became the haul road to Dam No. 1, Figs. 11 and 18. Quarry waste was dumped on the downstream face outside the downstream design slope of the dam to

FIG. 18 AERIAL VIEW DURING CONSTRUCTION LOWER BEAR RIVER DAMS NO. 1 AND 2

FIG. 19 AERIAL VIEW OF COMPLETED DAMS PRIOR TO FIRST FILLING.-LOWER BEAR RIVER DAMS NO. 1 AND 2

give a wider haul road and one that was available while the top small lifts were being constructed.

Placed Rock.—Fig. 11 shows the construction sequence. The lower placed rock of Stage 2 was placed from cranes operating from upstream of the cutoff or on the el. 5713 backfill. Stage 3 is an increment of construction where the placed rock was placed in lifts from a crane operating on the placed rock, dumped rock was then dumped and was caught by the placed rock.

Concrete.—Concrete was all placed by buckets handled by cranes. Cranes near the cutoff serviced the lower 80-foot slab and cranes on the crest serviced the top slab.

Performance

Dam No. 2 has had no face cracks, no leakage and no maintenance in its first five years of service. Settlement has been very moderate.

Settlement.—Settlement is shown by contours, section and tabulation on Fig. 20 and 21. The low settlement is associated with a dam, for which the Stage 1 main lift was used as a roadway to haul 800,000 cu. yds. of rock to Dam No. 1. Also, the main fill was completed in August 1951 and the water load applied one year and eight months later. The overfill of 176,000 cu. yds. of waste on a dam of 350,000 cu. yds. would have provided some resistance to horizontal deflection, but perhaps not to vertical settlement.

The movements as shown on the settlement contour drawings of Fig. 20 are much smaller than the movements of Salt Springs, but they have similar characteristics. The first year waterload settlement is greatest near four-tenths the height and the later aging settlement is greatest at the crest. Fig. 20B indicates that the aging settlement is greater near the left abutment. The dam being dumped from right to left abutment, the left abutment fill did not obtain the compaction of the drag of the dumped slope to the extent that the right abutment did. The crest settlement due to both water load and aging is about the same for Sections 13 + 35 and 18 + 15 even though the Section at Station 18 + 15 is twice the height. If the abutment profiles had been the same, more specific comments on the effect of the direction of dumping on the settlement could be made.

The sections and tabulation of Fig. 21 give a detailed record of the settlement during the first four years. The crest settlement of 0.21% for first year and total of 0.25% for four years, for the 140-foot high section at Station 15 + 75, is low.

Joint Movement.—The measurements across the 1 inch open vertical joints are not shown because there has been zero opening or closing of the joints. As has been commented on in discussing the Dam No. 1 joints, it takes a large force to close a vertical joint and the temperature stresses alone have not been great enough to cause the necessary crushing in and below the joint ribs. It appears that this dam could have been faced with a continuous slab without experiencing any cracking.

CONCLUSION

An attempt has been made to interpret and discuss the performance of the Salt Springs and Lower Bear River concrete face rockfill dams. However, the value of this paper is considered to be in the reasonably complete presentation of facts, from which the reader can draw his own conclusions.

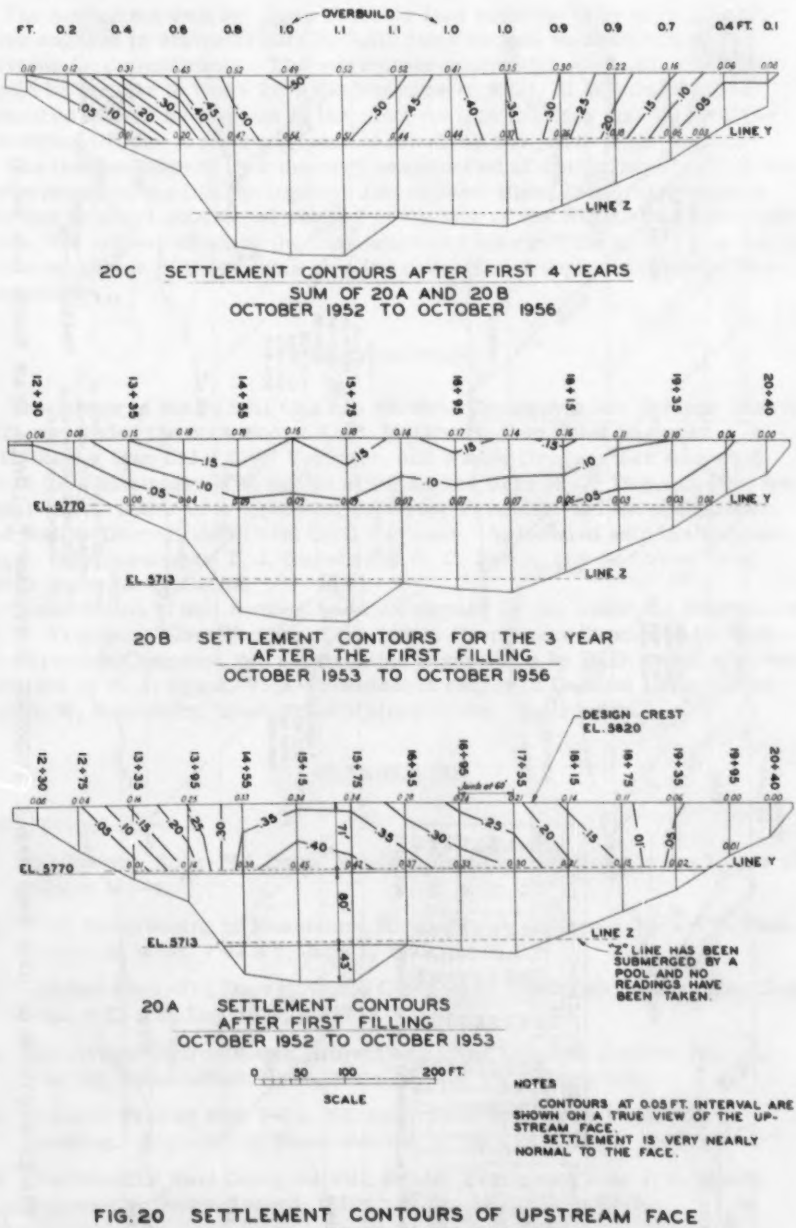


FIG. 20 SETTLEMENT CONTOURS OF UPSTREAM FACE
LOWER BEAR RIVER DAM NO. 2

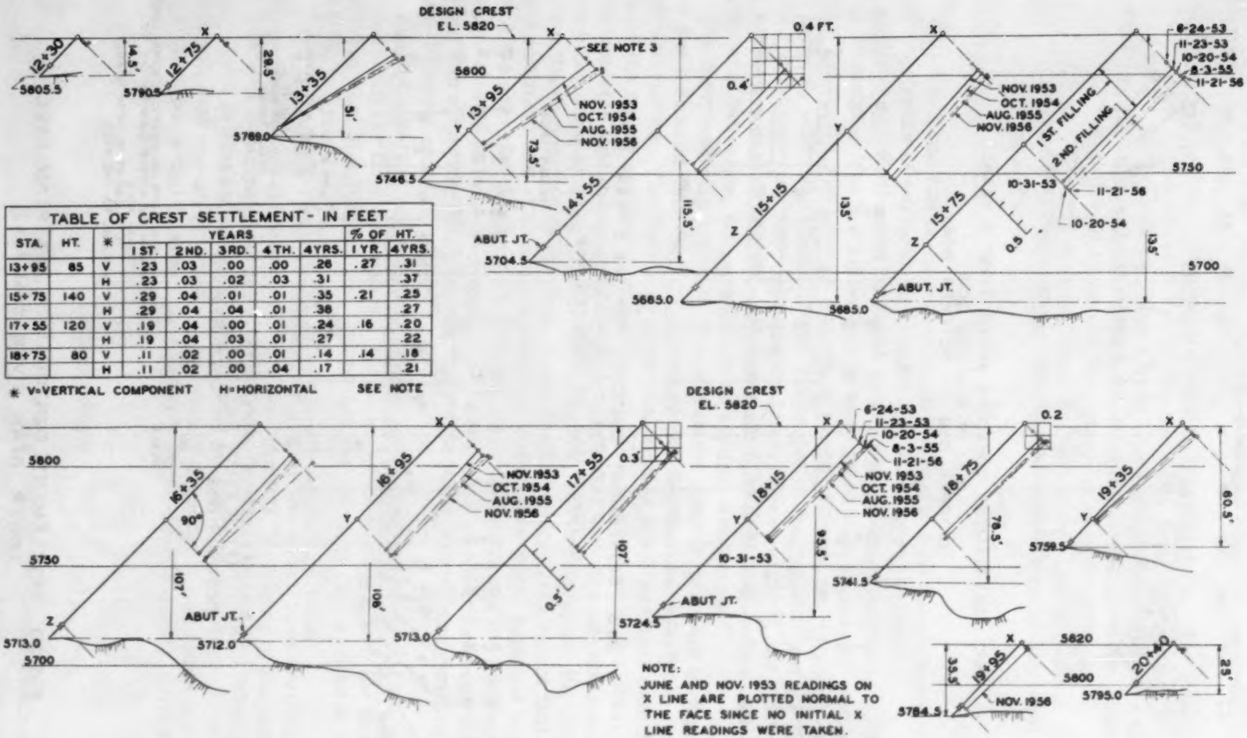


TABLE OF CREST SETTLEMENT - IN FEET

STA.	HT.	*	YEARS					% OF HT.	
			1 ST.	2 ND.	3 RD.	4 TH.	4 YRS.	1 YR.	4 YRS.
13+95	85	V	.23	.03	.00	.00	.26	.27	.31
		H	.23	.03	.02	.03	.31		.37
15+75	140	V	.29	.04	.01	.01	.35	.21	.25
		H	.29	.04	.04	.01	.38		.27
17+55	120	V	.19	.04	.00	.01	.24	.16	.20
		H	.19	.04	.03	.01	.27		.22
18+75	80	V	.11	.02	.00	.01	.14	.14	.18
		H	.11	.02	.00	.04	.17		.21

* V=VERTICAL COMPONENT H=HORIZONTAL SEE NOTE

FIG. 21 SECTIONS OF DAM AND TABLE SHOWING SETTLEMENT - LOWER BEAR RIVER DAM NO. 2

The settlement data for these concrete face rockfills is of value to engineers engaged in design of core rockfill dams as well as dams having impervious face membranes. The movements essentially normal to the face should be similar to those for a sloping core rockfill. It is noted that the measured lateral movements of the concrete face are less than those of the underlying fill due to the resistance of the slabs and joints to movement.

The lessons learned by a thorough examination of design, construction and performance of the Salt Springs and Lower Bear River Dams have made it possible to effect substantial savings in the cost of the Wishon and Courtright Dams,[29] without affecting the unquestionable safety of the dams. The design and construction of these dams is being covered in a separate paper in this Symposium.

ORGANIZATION

The design of the Pacific Gas and Electric Company's Salt Springs Dam in 1928 was under the direction of A. H. Markwart, then Chief Engineer, I. C. Steele, then Chief Civil Engineer, and Walter Dreyer, then Assistant Chief Civil Engineer. The design of the Lower Bear River Dams in 1950 was under the direction of I. C. Steele, then Vice President and Chief Engineer, and Walter Dreyer, then Chief Civil Engineer. Associated with the engineering on both dams were T. J. Corwin and G. C. Green, and on Lower Bear River Dams, J. B. Cooke.

Construction of Salt Springs was by Company forces under the direction of O. W. Peterson. Construction of the Lower Bear River Dams was by Utah Construction Company, and construction supervision by P. G. and E. Co. was directed by A. J. Swank, Vice-President in charge of General Construction and H. W. Haberkorn, Manager of Hydroelectric Construction.

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Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: THE DALLES CLOSURE DAM[a]

Robert J. Pope[1]
(Proc. Paper 1738)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

ABSTRACT

"Rockfill Dams: The Dalles Closure Dam" by Robert J. Pope. Construction of a closure dam for The Dalles project in 180-foot depth of water with flows up to 200,000 c.f.s. presented a problem solved by the design and construction of a rockfill structure, two-thirds of which was built under water without use of cofferdams.

INTRODUCTION

The Dalles Dam, located 192 river miles above the mouth of the Columbia River and about 3 miles above the City of The Dalles, is a multi-purpose

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1738 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4 August, 1958.

a. Presented at meeting of ASCE, June, 1958, Portland, Ore.

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FIG. 1 - View of Dam Site April 1951

project in the interest of power, navigation and irrigation. The ultimate capacity of generated power will be 1,743,000 kilowatts, slack water navigation is provided for over 20 river miles and minor irrigation and recreation benefits are also obtained. Construction of the project was started in February 1952, the closure dam was completed April 1957. Four of the turbines are currently in operation and completion of 14 turbines which will provide an installed capacity of 1,119,000 kw is scheduled for November 1, 1960. Normal pool elevation is 160 and average tailwater elevation 74, furnishing a power head of about 86 feet.

The aerial view of figure 1 shows the site for The Dalles Dam as it appeared in 1951 before start of construction. Bedrock at the site was formed as a series of superimposed basalt flows which are exposed as benches and cliffs on both sides of the river. The most significant structural feature is the Big Eddy fault which crosses the axis of the closure dam in the river channel and undoubtedly is responsible for orientation of the channel. The river flow is deflected from west to south at Big Eddy through a restricted outlet, then turns back west along a narrow channel for five thousand feet, then makes a second S curve to resume its westerly course.

A study of the site logically led to the location of the spillway designed to pass over 2 million c.f.s. and the powerhouse for ultimate development of 1-3/4 million kilowatts on the rock shelf which was skirted by the river channel on three sides. Plenty of room was available at this location for construction with a minimum of cofferdaming while the river continued to flow in its main channel. An aerial photo, figure 2, taken in March 1956 shows the construction as of that date. The navigation lock, spillway, non-overflow dam and powerhouse structures are essentially complete. The only missing major structure for the project to be a closure dam at the narrow outlet from Big Eddy.

General Plan

At the location selected for the closure dam, the river was restricted to a width of about 600 feet, the narrowest point in the vicinity. The original depth of the river in this area varied from 120 to 180 feet. Approximately one million cubic yards of rock from preliminary powerhouse excavation had already been placed in the closure area, reducing the depth of water at the axis to about 55 feet. Upstream the river bottom dips down into the depths of the Big Eddy pool which was 200 feet in depth with a bottom elevation 118 feet below mean sea level.

A rock and gravel embankment was the obvious choice for the type of structure at this location due to an abundant supply of these materials adjacent to the site. However, conventional construction methods, including cofferdams for dewatering the foundation presented difficulties which were readily apparent.

The general plan developed for constructing the closure dam consisted of first constructing a dumped rock diversion fill to divert the river flow through eight skeleton units in the powerhouse, then completing the closure dam by raising the dumped rock fill and placing an impervious blanket on the upstream face.

FIG. 2 - View of Construction March 1956

Schedule Limitations

The most difficult aspect of design of The Dalles closure dam was development of a plan which could be scheduled for completion within rigid time limitations imposed by the duration of low water season for diversion through the powerhouse, weather conditions and a minimum of interference with river navigation and fish migration.

The favorable low water season for construction on the Columbia River extends from October through February during which time the mean daily flow is approximately 100,000 cubic feet per second. Flows greater than 200,000 cubic feet per second have not occurred prior to November 7, and flows greater than 300,000 cubic feet per second have not occurred prior to December 23. All winter floods are of short duration. The longer duration spring freshets due to snow melt, starts its rise after April 1, generally peaks during June and has produced peak flows as high as 1,240,000 cubic feet per second. It was, therefore, necessary to design the diversion fill so that it could be constructed during the early winter months, during possible winter flood flows up to 300,000 cubic feet per second. It was also necessary to have the closure dam sufficiently complete to close the gates to the skeleton units of the powerhouse and raise the pool to pass the flow over the spillway prior to the spring freshet, as flood flows in excess of 300,000 c.f.s. through the powerhouse would threaten serious damage. This was because the structure could not be designed for summer floods.

Since barge traffic on the river is an important factor in the regional economy, it was necessary to hold to a minimum the time of closure to this traffic. Before construction of The Dalles Dam, navigation of the river for about 12 miles of non-navigable river had been bypassed through The Dalles-Celilo canal with its entrance just downstream of the closure damsite. The actual time of closure to barge traffic was less than 3 months. This was accomplished by permitting continued operation of The Dalles-Celilo Canal until necessary to close the gap for completion of the dam. A transfer pipeline was also provided from below the dam to the canal above to permit continued movement of petroleum products during the short period of navigation interruption.

A more-or-less continuous migration of salmon and steelhead up the Columbia River to spawning beds above the dam site occurs from late April through October. Since obstruction of these runs would result in considerable economical and recreational loss to the region, obstruction of the river channel had to be delayed until October and the pool raised by the following April to enable operation of the fish passing facilities. This would avoid the expense and difficult operations of temporary fish passage facilities.

Fortunately the favorable construction season with regard to river flows, navigation and fish migration coincided reasonably well. However, these restrictions imposed a 6-month schedule for the placement of almost 2,000,000 cubic yards of material in the main part of the closure dam during the winter season with severe winter weather and high river flows probable during the last 4 months. It was therefore necessary to facilitate construction in every possible way. Construction procedures were planned so that occurrence of normal winter floods or icing conditions would cause only minor interference with the schedule and emergency measures were planned to assure sufficient completion to allow closure of the powerhouse diversion even in the event of the occurrence of a record winter flood.

Diversion Scheme

Completion within the required time was dependent upon development of a diversion method which could be rapidly completed with certainty under all expected river and weather conditions. Previous river diversions of the Columbia and Pend Orielle Rivers had been end dumped using rock varying in maximum weight from 5 to 35 tons, or constructed with tetrahedons placed from overhead cableways. The use of heavy rock or similar construction methods were considered necessary during early planning for The Dalles diversion fill. However, further study indicated considerable advantage could be obtained by the construction of an adequate diversion channel to the skeleton units of the powerhouse. Excavation of a diversion channel 695 feet in width with a bottom elevation of 60 reduced the head required for diversion and actually permitted 25 percent of the river to flow through the 8 skeleton units of the powerhouse before the diversion fill was started. An increased width of diversion fill was also advantageous in that it reduced the gradient through the closure gap and in addition, provided adequate working width for the contractors operations on the fill. An increased width of diversion fill was economically feasable because the diversion fill was to be incorporated into the completed closure dam. Construction of a fill approximately 250 feet in width with an upstream leading edge created more favorable hydraulic conditions, by dissipating most of the energy at the upstream edge which permitted the smaller rock to be deposited downstream but within the width of diversion fill.

Because of the limited time available for construction of the diversion and unusual channel and hydraulic conditions, a series of model studies were conducted at the Bonneville Hydraulic Laboratory. The studies were made in a 1:40 scale model of the closure area. A number of variations in the construction method were studied including various rock sizes and different placement procedures. These studies varified the feasibility of constructing the diversion fill by end-dumping methods and demonstrated that a closure could be made using quarry-run rock having a maximum size of 1,000 pounds.

Blanket Design

Design of the upstream blanket sections was governed by the usual criteria regarding slope and foundation stability, seepage control and filter protection. Since construction time was of the essence, complicating refinements in section were avoided to the fullest possible extent and serious consideration was given to the practical limitations inherent in underwater work with respect to segregation of materials during placement, the difficulty of inspection, and the probable accuracy of underwater survey measurements.

Extensive explorations were conducted in search for materials suitable for blanketing the upstream face of the rockfill dam. These explorations revealed adequate supplies of poorly graded, sandy gravels; well graded, sandy gravels; and an abundance of non-plastic silts and fine sands. Silts and fine sands were considered undesirable for underwater placement because of flat slopes required and sensitivity to flow slides. Design of the blanket section and selection of materials was dependent on first determining the method for underwater placement of materials. Serious consideration was given to devising methods for placement which would minimize segregation and insure stable

construction slopes. Most of the methods considered had disadvantages of slow construction rates, high costs, or required special equipment which would be expensive, untested and unfamiliar to the contractors. The method considered most feasible was to place all underwater material by end-dumping at the water surface or slightly above except for a minimum width of upstream blanket which would be lowered in place. All underwater slopes were built on the natural angle of repose of the dumped materials except the final outside slopes of the completed dam. This method limited the choice of blanket materials to granular materials with non-plastic fines which would assume reasonably steep underwater slopes without incurring danger of large slump failures during construction. End dumping and building slopes on the natural underwater angle of repose had the advantage of low cost, fast placement rate and required a minimum of underwater survey control.

A typical cross section of the Closure Dam is shown on figure 6. The top of the dam is 295 feet above the upstream toe of the blanket material and the length of dam approximately 2,000 feet. The upstream slope is 1 on 2 above elevation 90 and 1 on 2.3 below elevation 90, the major portion of which was constructed underwater. The downstream slope is 1 on 1.5. Top width of the dam is 30 feet to accommodate road and railroad access to the upper deck of the powerhouse. A berm on the downstream slope provides access to the lower deck of the powerhouse. Base width is approximately 1,200 feet. The section below elevation 90, approximately 75 percent of the cross sectional area in the river channel, was designed to obtain the greatest possible advantage of the rapid placing rates made possible by end dumping methods.

Materials of the various zones grade in coarseness from quarry-run rock to sandy gravel. Each zone acts as a filter for the next adjacent upstream zone of finer material. Rock from preliminary powerhouse and cofferdam excavations which was placed in the closure area to approximately elevation 20 during the early stages of construction was dumped by truck from a barge anchored in the river for a pontoon roadway. The maximum depth of this fill was about 125 feet which left 55 feet depth of water over the fill at low river flow. The remaining quarry run rock and spalls for the embankment came from a quarry on the Oregon shore near the end of the abutment. The rock was a hard, durable basalt with a unit weight of 175 to 180 pounds per cubic foot. The rock was used in quarry run sizes which ranged from 200 to 300 pound average weight. Rock spalls were reasonably well graded with a 12-inch maximum size. Rockfill and spalls placed above the water surface were spread in layers 3 to 5 feet in thickness depending on the maximum size of rock and compacted with the spreading and hauling equipment.

Gravel was obtained from a river bar deposit about 3 miles upstream of the dam. Material from this source consisted of skip graded sandy gravel with a maximum size of about 4-inches and lacking material between the No. 4 and 30 mesh screen size. The gravel was used as it came from the pit, placed below water by end dumping off the edge of the fill and above water, spread in layers 18-inches thick, and compacted with 2 passes of a 50-ton rubber tired roller.

The upstream blanket was constructed of sandy gravel obtained from a high terrace deposit about 3 miles east of the site. This material was a well graded sandy gravel containing about 35 to 50 percent sand. The blanket materials were placed in the same way as the gravel except that the outer wedge below elevation 90 was lowered into place so as to prevent segregation and thereby insure that the coarse materials which accumulated at the base of the dumped material would be adequately blanketed.

Providing stability against sliding of a dam constructed entirely of granular materials in the manner described is a relatively simple problem. A safety factor of 1.0 against surface slides is inherent in the method of underwater construction of slopes on the natural angle of repose. Flattening the finished upstream and downstream slopes increased the minimum safety factor to 1.5 or better for any applicable condition of analysis. The safety factor of the upstream slope is further increased with a normal reservoir level due to the sloping upstream impervious blanket.

Laboratory permeability tests on the sandy gravel blanket material indicated a permeability ratio of from 12 to 20 x 10^{-3} feet per minute. This is many times faster than the permeability usually considered necessary for an impervious zone, but the computed seepage loss of 100 to 170 c.f.s. through the newly completed blanket was expected to decrease considerably with time as silt is deposited on the outer slope.

Foundation Exploration

Preliminary foundation explorations in the closure channel had been limited in extent due to the difficulties of underwater explorations, obvious exposures of solid basalt rock on each side of the channel and because it was expected that the high river velocities in the channel would keep the river bottom swept clean. This had been partially verified by a few difficult and costly inspections made by a team of divers in 1945. When more detailed information was required for final design, it was found that the river currents were too swift in many areas to obtain a reasonably complete examination of the river bottom by use of divers. A closed circuit television camera in a watertight case operated on cables from a barge was then utilized to survey the bed of the river.

The camera was operated in swift water by first lowering a heavily weighted anchor line to the river bottom. The camera was then lowered with slip rings attached to the anchor line. The camera was mounted in a tripod frame provided with lights for underwater illumination as shown on figure 3. Record photographs of the foundation conditions were made from the TV monitor screen. Riverbed materials were identifiable on the television screen as shown in the photograph of figure 4.

Considerable information was quickly and economically obtained by use of the underwater television camera. It was learned that approximately one-third of the foundation was covered by sand or gravel, and boulder deposits were found at the base of steep underwater cliffs. The upstream toe of the rock fill placed during powerhouse excavation was carefully mapped. Annual surveys by both TV camera and hydrographic surveys were conducted following each spring freshet to observe if any erosion of the fill had occured and the amount and nature of any deposition in front of the fill. The rock fill withstood four annual flood flows before completion of the closure without appreciable erosion and a maximum of 7 feet of coarse gravel was deposited adjacent to the upstream toe of fill. Underwater mapping with the TV camera eliminated areas of exposed bedrock from further exploration and difficult and costly drilling was restricted to gravel and sand areas. The next step was to determine the depth, gradation and permeability of the alluvial material. Considerable difficulty was experienced in obtaining suitable borings due to swift river currents and water depths of 100 to 120 feet. Drilling was accomplished by

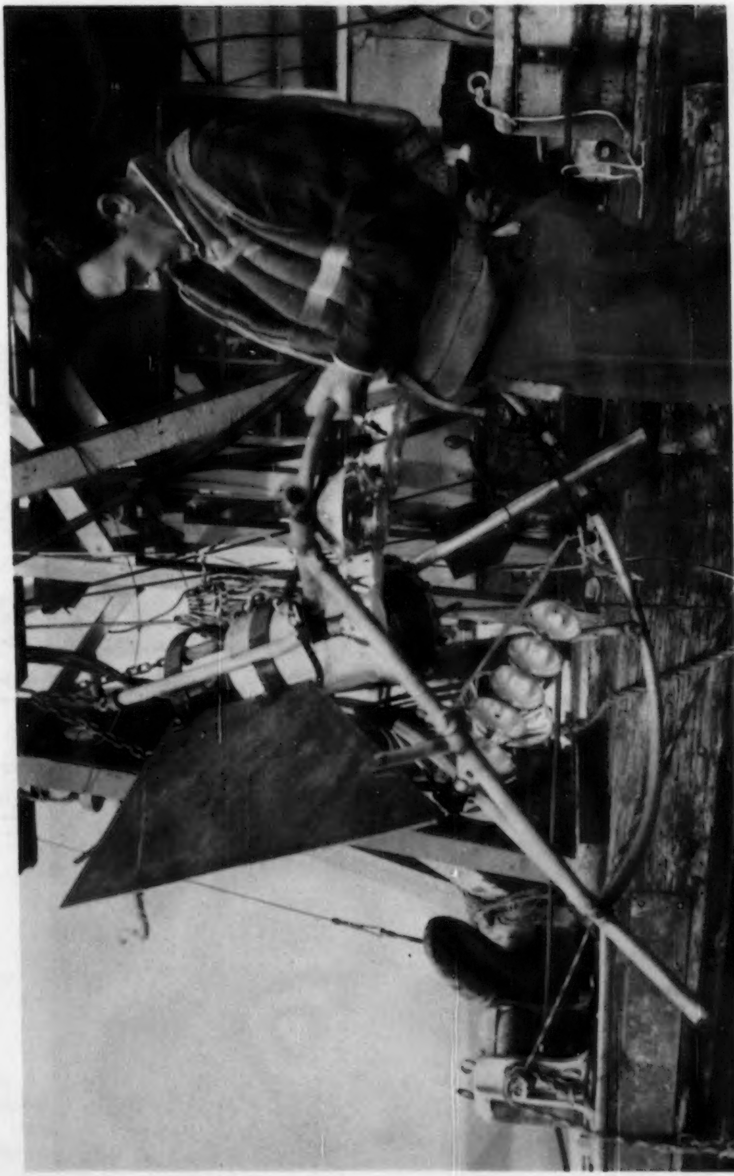

FIG. 3 - TV Camera for Underwater Use

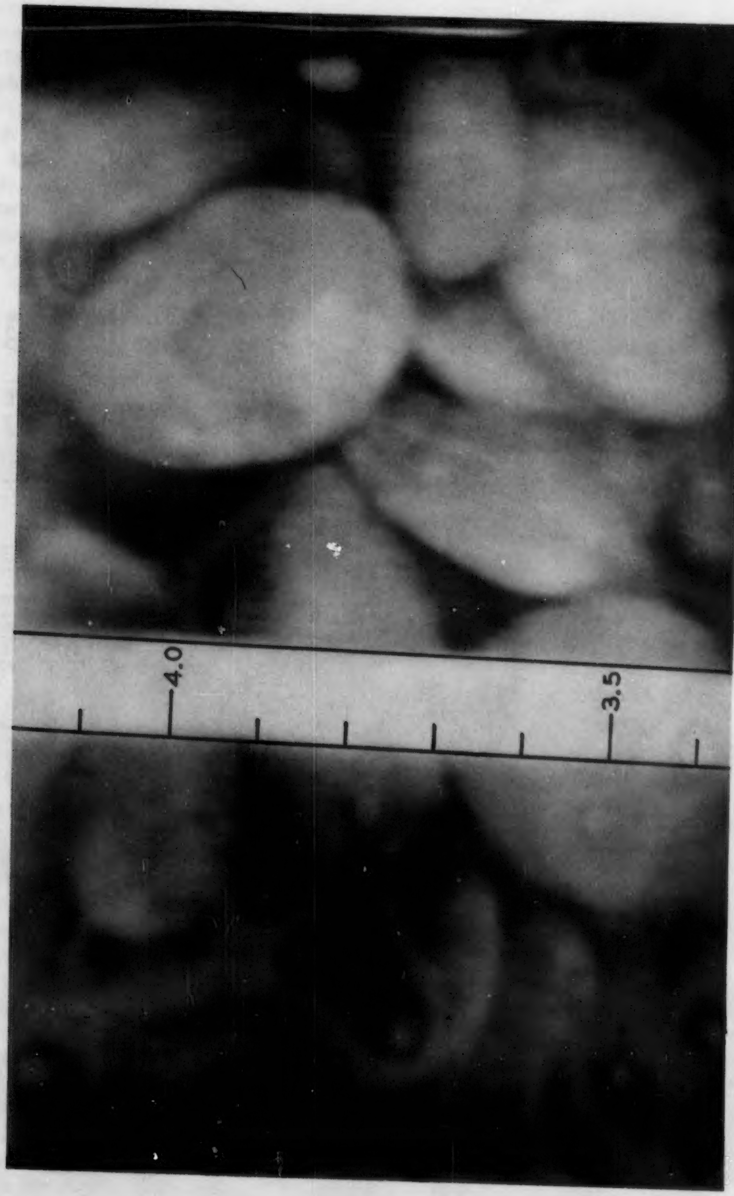

FIG. 4 - TV View of Gravel Bottom

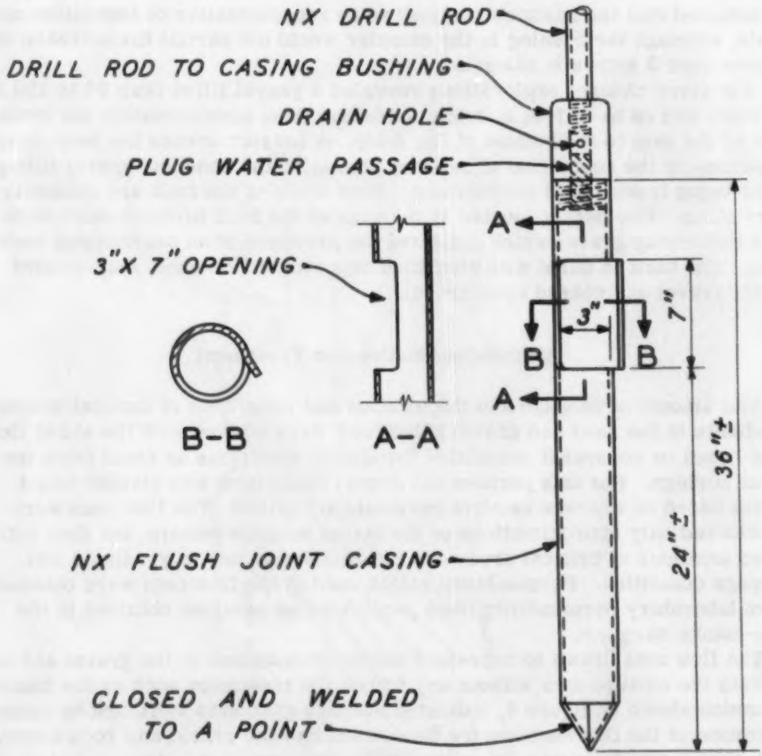

FIG. 5 - Side Intake Sampler

operating through the center well of a barge. An outer 6-inch casing, guyed near the bottom and midpoint, was lowered and seated into the riverbed. A 4-inch casing was lowered inside this and the boring and inside casing advanced to bedrock by the usual methods of chopping, washing and driving. Bedrock was cored with a diamond bit.

Samples of overburden sands and gravels were obtained as the 4-inch casing was withdrawn using a side-intake soil sampler which was shop built from a 3-foot section of NX flush joint casing as shown on figure 5. This sampler was lowered on the drill rods to the elevation at which a sample was desired and the 4-inch casing then raised until the bottom was above the opening in the sampler, but not above the top shoulder of the sampler. The sampler was then rotated slightly and withdrawn. A total of 23 borings were made in the river bottom of which 12 were sampled by the above method. In addition to the borings, the top of rock was sounded by probings to define the edges of the fault. Consistent results were obtained by the above sampling method and it

is believed that the samples obtained were representative of foundation materials, although the opening in the sampler would not permit the entrance of stones over 3 inches in diameter.

The river channel explorations revealed a gravel filled fault 90 to 130 feet in width and up to 80 feet in depth, extending from approximately the centerline of the dam to the bottom of Big Eddy. A barrier across the fault at approximately the centerline of the dam prevented the sand and gravel filling from being transported downstream. Rock walls of the fault are evidently very steep. One boring located at the edge of the fault broke through rock into underlying gravel which indicated the presence of an overhanging rock wall. The fault is filled with stratified deposits of fine sand, skip-graded sandy gravel and coarse open gravel.

Foundation Studies and Treatment

The amount of seepage and the location and magnitude of critical seepage gradients in the sand and gravel filled fault were studied with the aid of flow nets based on somewhat simplified foundation conditions as found from the river borings. For this purpose the gravel filled fault was divided into 4 strata based on average relative permeability ratios. The flow nets were considered only approximations of the actual seepage pattern, but they indicated locations of critical areas and the relative values of gradients and seepage quantities. Permeability ratios used in the flow nets were obtained from laboratory permeability tests performed on samples obtained in the side-intake sampler.

The flow nets drawn to represent seepage conditions in the gravel and sand beneath the closure dam without any foundation treatment such as the blanket extension shown on figure 6, indicated seepage gradients sufficient to cause migration of the fine sands in the foundation into the gravel and rock zones. Such piping would not greatly endanger the stability of the closure, but could result in subsidence of the foundation under the blanket and gravel zones sufficient to disturb the blanket and cause concentrated leakage which would be difficult to correct.

Corrective methods studied included an upstream blanket, placement of graded filters to prevent transportation of fines, a pile cut-off to rock and a partial cutoff trench. The plan of blanketing the area upstream from the toe of the dam was adopted as the most practical and economical for underwater construction. The blanket is an extension of the lowered-in-place portion of the blanket zone, and constructed using the same well graded sandy gravel and placed by the same method. The blanket extends upstream from the toe of the dam for a distance of approximately 250 feet. The seepage gradients under the blanket and gravel zones of the dam were effectively reduced, and the estimated quantity of seepage through the foundation was reduced from 260 to 110 cubic feet per second.

Construction

A contract for construction of the closure dam was awarded in October 1955. This allowed nearly a year for construction of haul roads to the borrow areas and quarry site and for construction of a portion of the dam at the left abutment. The diversion fill was started September 10, 1956 by dumping rock

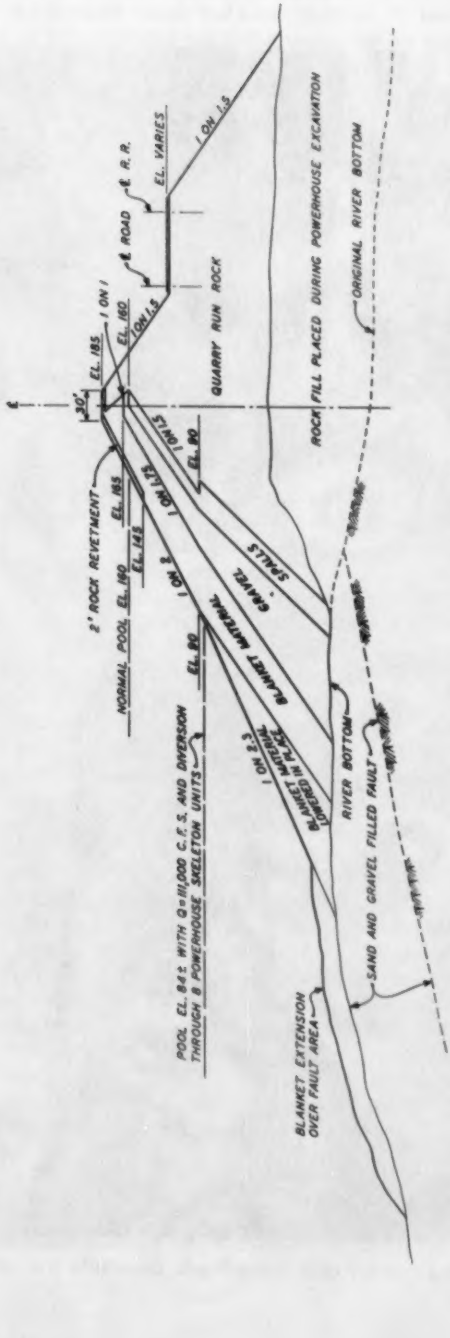

FIG. 6 - Typical Cross Section

FIG. 7 - Diversion Fill Oct. 15, 1956

in the main river channel from the Oregon shore. The actual diversion closely followed the method which had been verified by model study. A 240-foot frontal width of fill provided adequate space for an expedited rock placement program. The upstream edge of the fill was maintained slightly in the lead to minimize loss of rock outside the dam section as the model study had demonstrated. The final 25-foot width of river channel was closed in a period of 90 minutes on October 17, 1956 by concentrating all dumping at a 60-foot wide leading edge. During this critical stage of closure the river flow was 111,000 c.f.s. and a head of approximately 4 feet through the closure gap created velocities of about 12 feet per second. The photograph of figure 7 shows the diversion fill nearing completion.

End dumping of spalls and gravel followed advancement of the diversion fill as closely as river velocities through the remaining gap would permit. All material to be dumped under water was first dumped on the edge of the fill and then dozed over the shoulder which was maintained not more than 5 feet above the water surface. These precautions assured under water slopes on the natural angle of repose. No difficulty was experienced with shoulders breaking back or slides within placed material. The under water dumped angle of repose of the gravel averaged 1 on 1.8, blanket material 1 on 1.7, and rock or spalls 1 on 1.3. Material for the lowered in place blanket and upstream blanket extension was placed by use of 12-cubic yard skips lowered with a 100-ton derrick barge. The skips were not dumped until in contact with the bottom in order to minimize segregation during placement.

Under water inspections were conducted by divers at the completion of each stage of blanketing. Particular attention was given to all areas at which the blankets contacted steep under water slopes at the sides of the deep channel and adjacent to the Oregon and Washington shores. These inspections revealed unfilled rock overhangs and unblanketed talus slopes at the base of under water cliffs. Gravel was packed into the rock overhangs by jetting and additional blanket material was placed adjacent to the contact areas in the form of fillets to assure adequate sealing at these critical zones.

Under water surveys prior to start of construction were conducted mainly with sonic depth finders, but construction surveys by lead line for greater accuracy were feasible due to the existence of quiet water after diversion had been accomplished.

The closure dam was essentially complete in time for the scheduled filling of the reservoir. The pool was raised in two increments. On March 10, 1957 at 10:00 AM the 8 intake gates in the diversion powerhouse units were simultaneously closed, and in 4-1/2 hours the pool had raised approximately 40 feet to spillway elevation 121. On the following weekend, March 15 and 16, the spillway gates were lowered and the pool raised to elevation 155. Navigation was then restored through the new navigation lock and the fish passage facilities placed in operation in time for the spring salmon run. A full pool elevation of 160 was delayed until July 17 in order to complete necessary revisions on an upstream railroad bridge. Figure 8 shows the closure dam essentially complete with the pool at elevation 155.

Performance Record

The only observable leakage since completion occurred on the Oregon shore between the left abutment and the old ship canal. Leakage of about

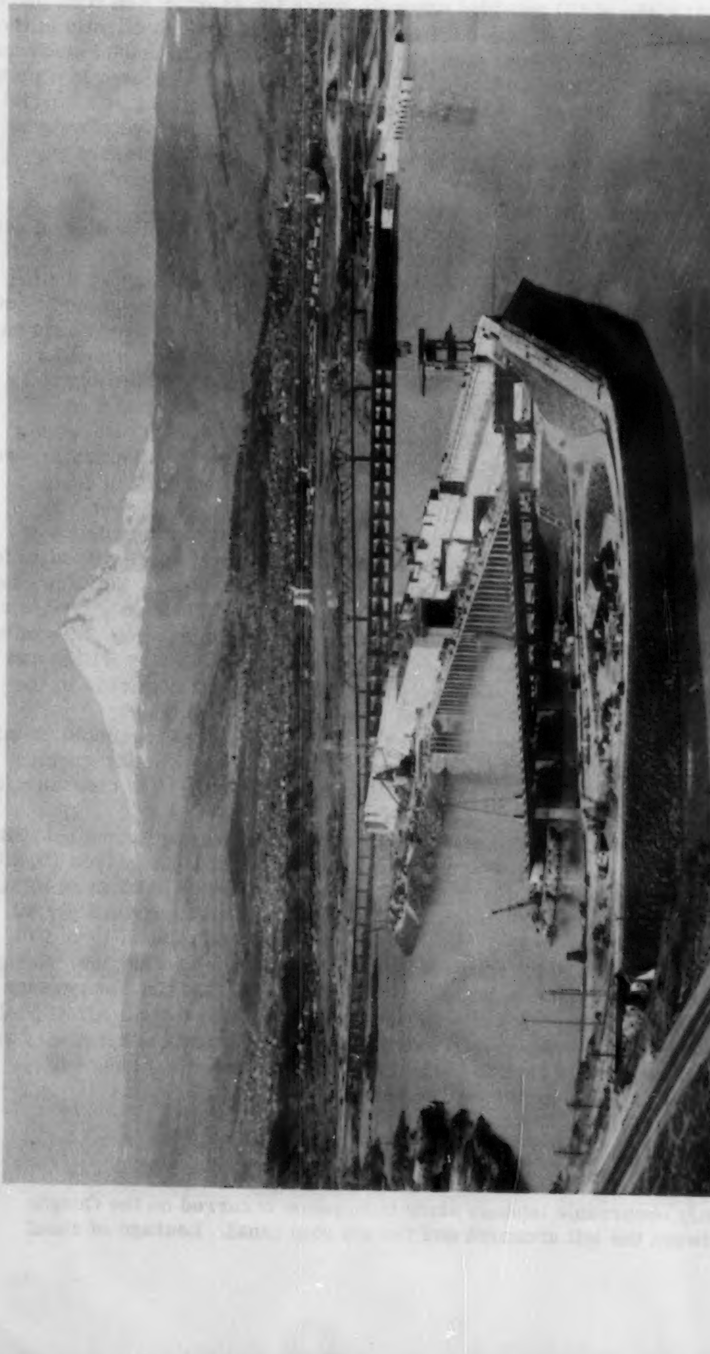

FIG. 8 - Completed Closure Dam Mar. 26, 1957

100 gallons per minute at the extreme end of the abutment was sealed by grouting joints in the rock which probably had been loosened by railroad construction. Considerable leakage was observed in the old ship canal when the pool was first raised, but no attempt was made to stop this leakage. It is probable that the total leakage through and under the dam is less than was anticipated because no salmon have been attracted to the pool at the downstream toe of the dam as would be the case if appreciable flow still occurred.

Settlement hubs along the top of the dam indicate about 0.5 foot of settlement occurred in a 3-week period during and immediately following raising of the pool. About 1 foot additional settlement has occurred in the following 14 months at a gradually diminishing rate. The settlement has been proportional to the depth of fill as would be expected and has been slightly less than anticipated.

ACKNOWLEDGMENTS

The Dalles closure dam was designed and constructed under the direction of the Portland District, Corps of Engineers, Colonel Jackson Graham, District Engineer; Ben L. Peterson, M. ASCE, Chief, Engineering Division; Paul Thurber, A.M. ASCE, Chief, Foundation and Materials Branch. Atkinson and Ostrander Company was the Contractor for construction. The writer is grateful to Mr. Ben L. Peterson, Chief, Engineering Division for review and construction criticism of this paper.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: REVIEW AND STATISTICS[a]

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(Proc. Paper 1739)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

This paper reviews present practices. It develops a definition, classification and terminology for such dams and for those using earth in addition to rock fill. It discusses the advantages of deck-type dams; also settlement and economy of rock-fill dams. In its appendix it lists rock-fill dams constructed in the United States and in foreign countries.

Unlike most, if not all, of the other papers forming parts of this symposium

Note: Discussion open until January 1 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1739 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at meeting of ASCE, Portland, Ore., June, 1958.

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on rock-fill dams, the authors treat herein the broader aspects of dam construction of that type.

Constructed as expedients during pioneer days in the West, several of those early, though smaller, rock-fill dams remain as functioning monuments to ingenuity and engineering enterprise. Since the close of World War II, rock-fill dams have increasingly received attention. Their importance is attested by the symposium of which this paper is a part.

There is of course ample justification for this interest in rock-fill dams. Primarily it lies in the resultant economy, but not in that alone. It is fair to say that various important dams constructed since that war would not exist were it not that selection of the rock-fill type made the construction economically feasible.

In former years a natural and perhaps laudable conservatism on the part of dam designers, seemingly justified by some failures, caused them to shrink from investing millions in large dams consisting mainly of what could be called a heap of loose rock, while in many instances a dam constructed of concrete could be had for nearly the same cost. Perhaps with some persons a like feeling still persists.

Chiefly responsible for the present-day favorable cost aspect of rock-fill dams is the fact that this type of construction is so well adapted to mechanization. Large-capacity quarry shovels and dump trucks, for loading and transporting the loose rock, permit construction rapidly, at a relatively low unit cost and with a minimum of personnel.

Also there is the fact that utilization of Nature's construction material, where plentifully available at or near the damsites, creates a situation wellnigh ideal for damsites which are not easily accessible. That is true, not only in the United States, where most of the readily accessible and otherwise desirable damsites are already in use, but also, as to dam-building in foreign countries. Thus over the last 10 years more high rock-fill dams have been constructed abroad than in our country.

A. Proposed Definition of Rock-Fill Dam

The name "rock-fill dam" is commonly applied to any dam in which rock fill is used. However, that name has been applied to various dams which, although utilizing rock fill, could more justifiably be designated otherwise, e.g., as earth dams or rubble masonry dams.

The term, "rock-fill dam," necessarily implies the use of two different materials, each having its own function. That is, a structure of rock fill alone will constitute an obstruction, but not a complete barrier, to stream flow. It can not truly constitute a dam.

An unsuccessful attempt to make rock fill alone constitute a dam occurred in the case of the Littlefield dam, on the Virgin River, Arizona, where natural silting was relied upon to seal the interstices of a rock fill and hence to provide impermeability. However, when construction had advanced to a height of 126 ft., or 11 ft. below the ultimate crest, a flood washed out the rock fill, well before the stage of overtopping had been reached (Western Construction News, Nov. 29, 1929, p. 618). In short, for a dam of rock fill, there is needed a second material, in order to make the structure impervious.

As yet no adequate definition of a rock-fill dam appears to exist, but the foreword to the papers of this symposium contains a suggestion believed to be worthy of development.

Criteria

It is proposed that the criteria for formulating an appropriate definition of a rock-fill dam be three-fold:

1. The material comprising the fill is loose rock, mainly taken at random, whether from a quarry or other source.

   Although the material is usually derived from quarrying operations, it is not the intention to exclude the use of natural rock slides or talus or rock from excavations for structures. In short, it is proposed that, to qualify as rock fill, the material consist of pieces of rock of various mixed sizes, obtained from whatever source is most convenient and economical.

2. The material is placed in the dam largely or entirely by dumping, regardless of whether the layers of such placing (usually referred to as "lifts") are relatively thin or thick and regardless of method of compaction—mechanical, sluicing or otherwise.

   This criterion would exclude dams constructed in the main of individually-placed rock. Such latter dams are actually rubble or dry-masonry dams and should not be called rock-fill dams.

3. The rock fill constitutes the major portion, i.e., at least 50%, of the maximum cross-section of the dam.

   This would exclude dams in which some other material, usually impervious (and usually rolled) earth, occupies more than half of the cross-sectional area. By "cross-sectional area" there is here meant the area of the maximum cross-section of the dam above a horizontal, or nearly horizontal, line which intersects the upstream and downstream slopes of the dam and which in general separates the man-made structure from Nature's materials upon which the dam is founded.

   For the purpose in point, material below such line, of whatever kind, would not be considered to be part of the pertinent cross-sectional area.

   Dams with very wide central earth cores, occupying more than half of the cross-sectional area, should be considered in reality to be earth dams, even though the material on each side of the earth core be rock fill.

   Likewise, dams sometimes referred to as being of a composite or combination type, that is, with an upstream section of rolled earth and a downstream section of rock fill, would be designated "rock-fill dams" only if the earth section were less in area than the rock-fill section.

   It is of course true that the proposed 50% dividing line is somewhat arbitrary and that rock-fill problems are just as real even though the rock fill may occupy less than 50% of the cross-sectional area. However, particularly for the purpose of distinguishing between earth-fill and rock-fill dams, the selection of the 50% point would be logical.

Proposed Definition

Based on the foregoing, the authors propose the following definition of a rock-fill dam:

> A rock-fill dam is one of which at least half the material in the maximum cross-section is comprised of loose rock placed by dumping.

In Section C hereof, there are illustrations of the application of such definition to various forms of 'fill' dams which utilize rock fill as a primary structural element.

B. Basic Types of Rock-Fill Dams

Rock-fill dams must be provided with an impervious portion or diaphragm —not necessarily a relatively thin one. Consequently the designation of basic type of rock-fill dam may logically be in terms of the location of the diaphragm in relation to the body of the dam. Thus there are two basic types, viz., the deck type and the central-core type. Of these, the former is believed to be the older.

1. Deck Type of Rock-Fill Dams

The general characteristic of this type of dam is that the impervious diaphragm, regardless of material used, is a deck on the upstream side of the body of the rock fill. The deck is relatively thin except generally when it is of earth without an upstream covering of rock.

Although the very highest rock-fill dams in the world are of the central-core type using earth, those of the deck type, in some cases nearly as high, are more numerous.

Deck Material

Various materials have been used in the construction of the impervious decks of rock-fill dams, with refinement to suit the taste of the designers or to follow contemporary habit and experience elsewhere.

It is not within the scope of this paper to discuss the details of construction of such decks; that has been done in other papers constituting parts of the symposium. A mere summary will suffice:

Wood

Planks are nailed onto wooden beams embedded in the upstream surface of the rock fill, which surface generally has been smoothed with a coating of portland cement mortar.

There are several dams with such decks of wood. J. D. Galloway in his paper on rock-fill dams (Trans., ASCE, Vol. 104, p. 18) stated that one of the advantages of wooden decks is flexibility. Existing examples are:

Hillside dam, 80 ft., California, 1904
Cucharas dam, 125 ft., Colorado, 1911
Saddle Bag dam, 47 ft., California, 1921
Torron dam, Sweden, 1936

Steel

Steel plates, provided with expansion joints, are fastened to steel beams embedded in the surface of the rock fill, similarly smoothed by means of cement mortar.

Details are described in "Steel dams," by O. E. Hovey. Examples of steel-deck dams are:

Skaguay dam, 70 ft., Colorado, 1901
Goose Neck dam, 210 ft., Colorado, 1910
Penrose-Rosemont dam, 100 ft., Colorado, 1932
Pego do Altar dam, 213 ft., Portugal, 1949

Earth (see Section C hereof, illustrative cross-sections 2 and 3).

The impervious 'diaphragm' is comprised of earth fill, deposited (with appropriate transition zones) on the upstream side of the rock fill, then rolled

for compaction and impermeability. This type utilizes solely Nature's materials, frequently in unaltered form, as found at or near the damsite.

Admittedly this type of dam is ordinarily not thought of as a deck type. However, functionally such use of earth fill is believed to compel classification as a deck type.

This is true even where additional loose rock is deposited over the upstream or outer face of a relatively thin, rolled-earth 'diaphragm,' not merely to protect the earth from wave action but also to ensure its stability. In such dams the use of the presumably more expensive of the two materials is minimized. Dams of this logically evolved sub-type (see Section C, cross-section 2) are frequently referred to as having a "sloping earth core." The principal exponent in North America of this sub-type has been James P. Growdon, M. ASCE.

Examples of dams with this relatively thin form of rolled-earth deck are:

Sillre dam, 26 ft., Sweden, 1933
Nantahala dam, 250 ft., North Carolina, 1942
Kenney dam, 324 ft., Canada, 1952
Brownlee dam, 395 ft., Idaho, under construction

The dams with thicker sections of earth (see Section C, cross-section 3), which have no outer covering of rock (beyond ordinary rip-rap) and which, by reason of the proportions of cross-sectional areas, must be classified as rock-fill dams, are likewise in essence of the deck type. In these dams the upstream slope of the rock fill usually averages steeper than the natural one of dumped rock on the upstream face of the rock fill.

Examples of dams with this relatively thick form of earth deck are:

Ashton dam, 60 ft., Idaho, 1917
Inland dam, 195 ft., Alabama, 1938
Barre Falls dam, 62 ft., Massachusetts, u.c.

In rock-fill dams involving earth decks—as also in the cases of those with central cores of earth—there must be transition or 'filtering' zones in which there is gradation from rock fill to earth.

In this connection it may be noted that similar dams, with the thicker earth fills on the upstream side, have been constructed by the Bureau of Reclamation, but with the downstream portion of the dams comprised of gravel and other pervious material instead of rock fill.

Portland Cement Concrete

The deck material most commonly used has been portland cement (p.c.) concrete. At present, the usual practice is to cast the concrete in panels upon—largely integrally with—a relatively smooth base formed by individually placed pieces of rock over the rough upstream surface of the rock fill proper.

That base is comparatively costly, but serves added purposes, of which the most important and certain is that of providing an upstream slope steeper than the natural slope of dumped rock. This reduces volume as regards both the p.c. concrete and the rock fill itself.

However, there is also a p.c. concrete deck which, apparently without any ill effect, has been placed upon a thick layer of loose, washed gravel, overlying the rock fill proper, namely, in the case of Cogoti dam, Chile (Engineering News-Record, Nov. 5, 1931, p. 725).

In the United States the principal constructor of rock-fill dams with p.c.

concrete decks has been Pacific Gas and Electric Company in California.

Prominent examples of dams with decks of p.c. concrete are:

Dix River dam, 275 ft., Kentucky, 1925
Salt Springs dam, 328 ft., California, 1931
San Gabriel #1 dam, 300 ft., California, 1938
Pinzanes dam, 220 ft., Mexico, 1956
Courtright dam, 310 ft., California, 1958

Asphaltic Concrete

The suitably prepared upstream face of the rock fill is paved with several layers of asphaltic concrete so as to form the impervious deck.

The only examples of dams with decks solely of asphaltic concrete appear to be:

Montgomery dam,* 113 ft., Colorado, 1957
Smith Mountain dam, 235 ft., Virginia, under construction.

Asphaltic concrete, as a material for the construction of decks for dams comprised of rock, has been used in foreign countries, although not in the form used at Montgomery dam or to be used at the Smith Mountain dam. In no case is the deck known to be solely of asphaltic concrete. Some of such dams are of rubble masonry.

Following are examples of those foreign dams which are known to be true rock-fill dams:**

Caritaya dam, 125 ft., Chile, 1935
Genkel dam, 140 ft., Germany, 1952
Henne dam, 200 ft., Germany, 1954
Iril-Emda dam, 246 ft., Algeria, 1954

2. Central-Core Type of Rock-Fill Dams

Central-core, rock-fill dams are those having the impervious portion or diaphragm located at or near the vertical center line of the body of the rock-fill dam. Further subdivision of this type may be made depending on the nature of the core, including its material, e.g., those with relatively thin, central core-walls, generally of some material other than earth, and those with relatively thick central cores, of earth.

Central Core-Walls

The thin, central-core wall is probably a descendent of very old earth dams and dikes, including Dutch dikes constructed several hundred years ago.

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* Detailed description appears in a paper by the authors, entitled: "Montgomery Dam—Rockfill with Asphaltic Concrete Deck" (Journal of Power Division, ASCE, February, 1958, Paper 1556). See also a paper presented by them at the February 1958 Annual Meeting of Association of Asphalt Paving Technologists, Montreal, Canada, entitled: "Laboratory Investigation of Asphaltic Concrete—Montgomery Dam, Colorado." The engineers for the owner, the City of Colorado Springs, Colo., were Black & Veatch, Consulting Engineers, Kansas City, Mo., to whom the office of F. W. Scheidenhelm, including the co-authors of this paper, served as consultants.

** For more extensive listing, see Journal of Power Division, ASCE, February 1958, Paper 1556, p. 10.

At a time when compaction of earth was accomplished as well as possible, but, as compared with present day means, in a primitive manner, the controlling thought was that a dam or dike constructed of earth alone could not surely be made watertight and that the mere insertion, vertically, of a thin wall of impervious material, approximately along the center line, would make it so.

Because earth slopes as flat as 1 (vertical) on 4 or 5 (horizontal) were then customary, many such dams were successful. However, when economy demanded steeper slopes, now and then some bitter experience resulted.

The use of thin, central core-walls in earth dams of whatever height has practically disappeared and their use for rock-fill dams is becoming very rare, even for relatively low dams. In both cases, the basic reason may be that the material of the fill upstream from the thin, central core is completely submerged and in any case contributes nothing to the stability of the dam. In fact, it adds to the horizontal water pressure against the core-wall and the downstream portion of the fill and thereby reduces the margin of safety.

In such central core-wall, rock-fill dams as exist, the material for the core-walls has been reinforced concrete, plate steel and wood or combinations thereof. None of these materials combines strength, flexibility and durability to a degree sufficient to warrant burial inaccessibly in the interior of a dam.

Existing examples of central core-wall, rock-fill dams are:

East Canyon dam, 93 ft., Utah, 1899
Crane Valley dam, 145 ft., California, 1910
Smith dam, 43 ft., Colorado, 1916
Silverwood dam, 80 ft., Australia, 1928.

Central Cores of Earth

A major improvement, particularly for high dams, is the use of a thick, rolled-earth, central core. It is of course still true that the completely submerged upstream rock fill adds to the horizontal water load against the body of the dam proper (core plus downstream rock fill). However, the thick earth core, together with adjacent rock fill, lends itself more readily to stability computations, approximating those usually made for conventional gravity-type dams of p.c. concrete. The rock-fill sections confine the steep earth slopes (with their appropriate intervening zones of transition from coarse rock to earth).

The highest rock-fill dams are of such construction. Examples of rock-fill dams with central cores of rolled-earth are:

Mud Mountain dam, 425 ft., Washington, 1948
Ambuklao dam, 420 ft., Philippines, 1955
Derbendi Khan dam, 410 ft., Iraq, u.c.
Trinity dam, 537 ft., California, u.c.

Hydraulically-placed or sluiced-earth fill, instead of rolled-earth fill, has been used for the construction of the impervious interior of at least two dams, which, because of the 'shoulder' of rock on each side of the hydraulic fill, have the appearance of rock-fill dams. These are the El Capitan (California) dam and the Cobble Mountain (Massachusetts) dam. However, under the definition of rock-fill dams, i.e., because of preponderance of earth, each must be classed as an earth-fill dam.

C. Classification and Terminology

There are few structures concerning which there is so much confusion in terminology as is true of rock-fill, particularly those in which earth is used. This is evident in casual references but also in more carefully considered correspondence and in the pertinent engineering press.

In the absence of a cross-sectional sketch of a given dam and of accepted terminology, it may be impossible to determine just what type of dam is involved. Frequently the desire for brevity results in ambiguity or vagueness. Surely such characterization as "a rock fill earth fill" structure lacks the preciseness which is generally typical of an engineer.

The authors believe that the illustrated classification, with terminology, below may be as helpful to others as it has been to themselves. It is in accord with the definition proposed at the end of Section A hereof and with the descriptions contained in Section B.

Mention and diagramming of dams solely of earth fill are omitted, due to absence of rock fill and as involving no possibility of confusion.

While not insisting that the following represents the optimum, the authors do state that it represents the results of earnest effort in that direction and they propose that it be considered for adoption as an outcome of this symposium.

D. Advantages of Deck-Type of Rock-Fill Dams

The deck type of rock-fill dam has various advantages not possessed by the central-core type. The more important of those advantages may be described briefly as follows:

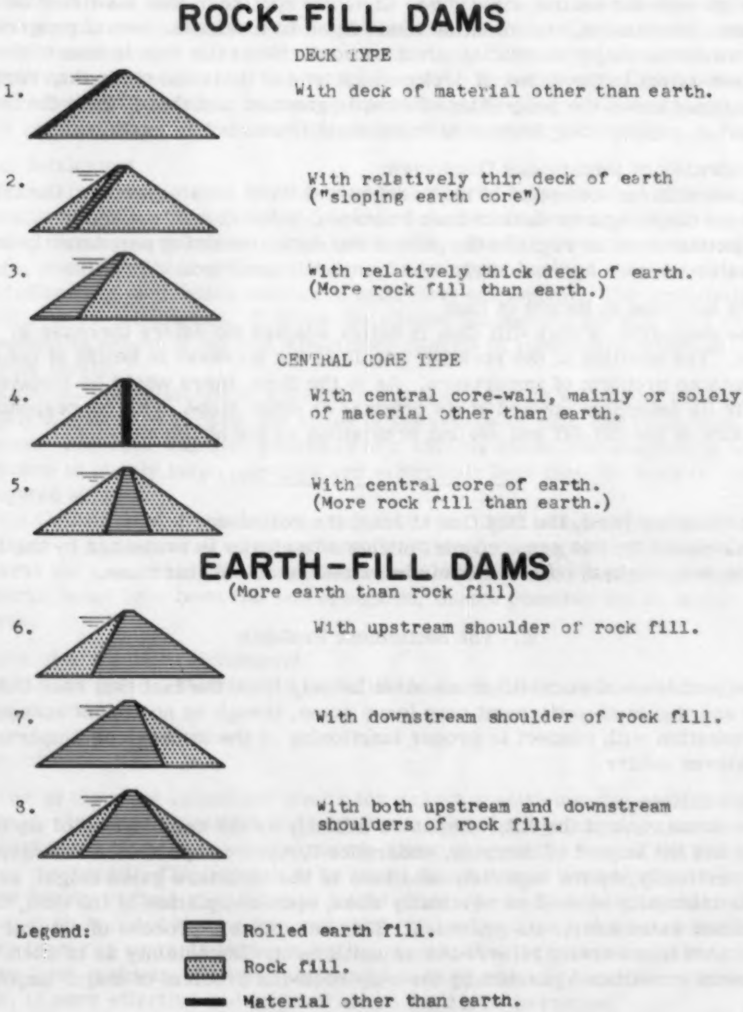
Greater Margin of Safety

Stability of the deck-type dam as a whole against failure by sliding or by overturning is amply assured, largely because no substantial amount of uplift pressure can be exerted. Particularly is this true of dams with decks of other than earth fill. The factor of safety in these respects is very large. On the other hand, in the case of a dam of the central-core type, none of the rock upstream from the core contributes to safety against sliding or overturning—which, however, is by no means the same as saying that by reason of this fact a rock-fill dam with a central earth-core will not be amply safe.

Greater Tolerance of Leakage

Ordinary leakage through the foundation or the deck can not endanger the dam, for the rock-fill is essentially free draining. Even some overtopping, with flow into the rock fill, is tolerable, though of course not desirable.

Because, with the exception of thick, rolled-earth decks, the area of grouting is at or relatively near the upstream edge of the dam, grouting and the major rock-fill construction can proceed simultaneously, though of course not in the same location. Thorough grouting is a time-consuming process, the extent of which generally can not be fully foreseen. Construction delay can be minimized where grouting and dam construction can go on at the same time. Moreover, because ordinary amounts of leakage through the rock foundation can not endanger the dam and where there is no material economic loss by reason of such leakage, grouting can be somewhat less than perfect.


ROCK-FILL DAMS
DECK TYPE
1. With deck of material other than earth.
2. With relatively thin deck of earth ("sloping earth core")
3. With relatively thick deck of earth. (More rock fill than earth.)
CENTRAL CORE TYPE
4. With central core-wall, mainly or solely of material other than earth.
5. With central core of earth. (More rock fill than earth.)
EARTH-FILL DAMS
(More earth than rock fill)
6. With upstream shoulder of rock fill.
7. With downstream shoulder of rock fill.
8. With both upstream and downstream shoulders of rock fill.
Legend:
Rolled earth fill.
Rock fill.
Material other than earth.

Rapidity of Construction

Except for the rock immediately underlying the deck, the construction of the main or sole rock fill, as the case may be, is unbroken, contrary to the situation with the central-core type. Once the routine of that construction has been established, the situation lends itself to maximum rate of progress, with correspondingly beneficial effect on cost. Naturally this is true to the greatest extent in the cases of decks comprised of material other than earth—for in those cases the proportion of rock is greatest and there is not the somewhat complicating feature of transitions from rock to earth.

Accessibility of Impervious Diaphragm

Again with the exception of earth decks and their rock coverings, the impervious diaphragm or deck remains accessible for inspection and repair. This is true even as regards the part of the deck remaining permanently under water where a hydroelectric development is involved.

Future Increase in Height of Dam

The deck-type of rock-fill dam is better adapted for future increase in height. The addition of the rock fill requisite for increase in height of course presents no problem of importance. As to the deck, there would be involved merely its extension upward at its original or other slope, with corresponding extension of the cut-off and sealing provisions up the abutments.

On the other hand, the fact that at least the rolled-earth form of the central-core type has some countervailing advantages is evidenced by the fact that the very highest rock-fill dams in existence are of that form.

E. The Settlement Problem

The problems of rock-fill dams stem largely from the fact that rock fills settle and that such settlement may leave some, though by no means unresolvable, question with respect to proper functioning of the impervious diaphragm, of whatever nature.

General Settlement

The loose rock of the fill, compacted initially by the combination of its own weight and the impact of dumping, undergoes further compaction, first essentially vertically, by the superimposed load as the structure gains height, and later horizontally as well as vertically when, upon completion of the dam, the impounded water exerts its pressure. This unavoidable process of natural compaction is generally referred to as settlement. Uncertainty as to such settlement constitutes practically the only rock-fill problem of major importance.

Loose quarried rock has irregular surfaces and edges which come to rest against each other in the process of dumping. The gradual increase in load causes these edges or points of contact to spall or crush and hence there is some re-adjustment within the fill. This process continues until the contacts between the pieces of rock have become sufficiently blunt to transmit the load without further spalling or crushing. The resulting settlement appears to continue, at a gradually diminishing rate, for a few years after completion of the structure. Settlement of this type may be assumed to be proportional to

the height of the rock fill above its foundation and to be present throughout the fill.

Surplus fill, sometimes referred to as "camber," and a slight convexity, upstream in plan, are usually provided to compensate for such settlement. Obviously convexity downstream in plan should be avoided because fill settlement or movement in a downstream direction, such as would be due to water pressure, would tend to pull the rock fill apart. Relief dam in California, constructed in 1910, has a convexity downstream in plan; a crack developed in its reinforced p.c. concrete deck, at the center of the curve.

Local Settlement

Probably more serious is what may be referred to as local settlement. When, in the process of dumping, pieces of rock become lodged in such a manner that there are relatively large holes within the fill, even the small adjustments, resulting from spalling of edges or points of contact, may cause partial collapse of such holes and hence additional settlement. The cumulative effect may be substantial. It is not distributed throughout the fill, may be independent of height of fill and sometimes results in subsidence which is strictly local.

Although the existence of excessively large openings within the rock fill is largely a matter of chance, the shape of the pieces of rock has important influence. For rock which is generally of a cubical shape, the chances of occurrence of unduly large openings are materially less than for rock of flat, elongated shape.

Yet Genkel dam, Western Germany, 140 feet high, is reported to be constructed of slate quarry waste, accumulated over many years of operation of several such quarries (Bitumen, December 1953, p. 204); in that instance the material must have been flat and elongated, though possibly not in large pieces.

Means of Minimizing Settlement

Although some settlement of rock fill is unavoidable, means are available to minimize it.

Sluicing

One of the most important means for reducing settlement is to direct jets of water, in large quantities and under relatively high pressure, upon and against the rock as it is being dumped; likewise as to the face of the fill between dumpings. The purpose is to cause the pieces of rock to come to rest more nearly in final position than would otherwise be the case. At the same time, such sluicing moves fine material ("fines"), of, say, 4 in. and smaller, away from the contacts between larger pieces, thus in general resulting in more firm contacts. Sluicing is usually done by means of nozzles of monitors, is very effective and in general can hardly be overdone.

Some text books state that such sluicing is done to wash the fines into the interstices of rock already placed. However, the authors suggest that such is the result rather than the beneficial purpose of sluicing. Actually, under sluicing, these fines can not go elsewhere than into such interstices and any resulting beneficial effect, as concerns later settlement, is at best not important in degree. That is, the sluicing process can not be expected to compact these fines to such extent that the result will be to resist materially, if at all, the settlement of the surrounding rock fill. On the other hand, in the

construction of Malpaso rock-fill dam, Peru, 1936, fine material (gravel and sand) was brought separately to the damsite for the sole purpose of being sluiced into interstices (Compressed Air Magazine, July, 1939).

Limitation of Proportion of Fines

Restriction of the permissible amount of fines in rock-fill material is very important. Whereas rock fill may be expected to have voids of the order of 35%, the proportion of fines in the material clearly should be far less; otherwise, by reason of intervening fines, the larger pieces of rock may lack the desired direct contact with each other.

Moreover, the combination of an excess of fines and liberal sluicing may result in the filling of the lower interstices to such extent and with sufficiently fine material as to bar or at least retard drainage within what should be a free draining rock fill. In the case of a deck-type dam, there could thus result the danger that water, draining from foundation rock or abutments or even leaking through the deck, would be ponded upstream from such interior barrier and in turn would cause reverse pressure, against the under side of the deck, at a time when the reservoir is drawn down. In the case of Montgomery dam, with the foregoing in mind, it was prescribed that a limited thickness of dumped rock, under and near the deck, should be free of material of less than 3-inch size. Also positive, i.e., open-joint, pipe drainage was provided along and from the bottom of that zone of screened rock.

Specifications for the body of the rock fill should prescribe the maximum permissible proportion of fines and provide positive means of testing and enforcing compliance. The specifications for the recently completed Montgomery rock-fill dam aimed to limit to 5% the proportion of fines (material less than 4-inch size), but at the maximum tolerated 10%. There was liberal sluicing. Settlement so far has been gratifyingly low.

Methods of quarrying may greatly affect the proportionate product of fines. Random blasting and in some cases the coyote-hole method may result in cheaper quarrying but not necessarily in an appropriate rock product. Blasting techniques worked out experimentally at the specific quarry site and for the kind of rock involved are more likely to minimize the output of fines.

It has been suggested that elimination of an excess of fines remaining in the run-of-quarry output might be accomplished by the use of (dump) trucks with slotted bottoms, thus permitting fines to be shaken out of the truck-loads enroute from quarry to dumping site. However, the authors know of no instance where such method has been carried out.

Supervision

Because of the very nature of a rock fill, the temptation is to assume that all that is necessary, to attain an appropriate ultimate result, is to quarry, haul and dump the rock, together with such sluicing as may be prescribed. However, the rock piled up on the quarry bench is not pre-tested, pre-examined as frequently is earth from borrow pits or pre-graded as are aggregates for concrete. All that is known is that the larger part of the pile is satisfactory and utilizable and that the rest, if used, would contribute toward inferiority.

Clearly, therefore, the price of good rock fill includes adequate supervision and inspection, especially at the quarry bench where loading of trucks takes place and at the dumping and sluicing locations. It need hardly be

suggested that such inspection is at least as important during night shifts as during day shifts.

Special Compaction

In a sense, a rock fill, given sufficient time, especially under water load, is self-compacting; yet, if no effort is made to minimize settlement, it may become unduly or, in extreme cases, intolerably large.

In the United States and Canada, rock fills for dams have been placed mainly in relatively high stages or lifts. Although economy of construction enters, it seems fair to say that a principal consideration is the belief that construction in high lifts is conducive to minimizing settlement. Without doubt, there results some compacting effect, due to the longer distance of roll of the larger pieces of rock down the face of a relatively high lift.

Then there are those who seek to accentuate initial compaction by means of impact due to greater height of fall of the rock, e.g., from loaded skips carried by cableway.

In contrast, there are cases (largely European) where, apparently for the attainment of like end, rock-fill placement is in relatively low lifts. Under that method, considerable compaction occurs within the shallow lift by reason of the repeated travel over the surface of the lift or layer by heavily-loaded trucks, transporting rock to the dump face. Given sufficient shallowness of layer, it would seem that vibratory rollers might be effective for the purpose. (On the other hand, the resultant creation of additional fines, due to attrition, is of itself undesirable.)

Various special means for obtaining initial compaction, or at least expediting settlement, result in additional cost which must be weighed against the advantage obtained.

It would be of interest and undoubtedly of substantial help for future rock-fill dam construction if discussions of symposium papers were to contribute pertinent information and views supplementing such as may be contained in the individual papers.

Diaphragm Flexibility—a Requirement

Settlement of rock fill can be minimized but not completely eliminated. One must therefore be reconciled to deal with the effect of such settlement as does take place. Primarily this requires that the impervious diaphragm be without danger of rupture.

Presumably the need for pertinent provision is less in the case of rock-fill dams of the central-core type than in those of the deck type. However, as hereinabove indicated (Section B), at least in the cases of central core-wals (material other than earth), the results have not always been satisfactory.

As to the behavior of central cores of earth, information is by no means extensive or positive—largely because the core is buried and there is no manifestation of behavior unless such be unfavorable, let alone resulting in failure.

As for decks, cracking or other forms of failure due to insufficient flexibility are of record in the cases of materials other than earth. Regarding the performance of decks of earth, particularly relatively thin earth-decks (sloping earth-cores), just as in the case of central cores of earth, little is known in the pertinent respect beyond the fact that there have been no failures.

Decks of portland cement concrete undoubtedly have substantial flexibility where relatively small, jointed panels are used. Possibly the same is true where there are used larger panels which are laminated, i.e., composed of

thin slabs. In the cases of decks where comparatively thick slabs of large area are used, question may arise. However, it appears that such construction has adequately met the needs of the cases where used—due perhaps to the fact that the slabs are underlain by what are in effect inclined walls of dry rubble masonry.

As for the newer deck material, asphaltic concrete, experiments and experience are believed to warrant the view that, within reasonable limits of temperature and deformation, that material will adjust itself by flow, without rupture, to maintain contact with its supporting base.

In relation to the minimizing of the effect of settlement, too, it is hoped that discussion of the symposium papers will bring out pertinent experience and facts, as well as views, not contained in papers as submitted.

F. Dams of Greater Height and Economy

The following is submitted by way of specific contribution toward the announced purpose of the symposium.

Greater Height

Dams of a given type and height having been constructed and in operation successfully, it may reasonably be assumed that, under equally favorable conditions, dams of like type and somewhat greater height may be constructed and operated successfully. However, the greater the height for which there is precedent, the more the need for caution in taking the step to still greater height—not overlooking the fact that in general the greater the height of a dam, the greater the potentiality of damage in case of failure.

Always there is the necessity for careful analysis and comparison of the pertinent conditions at the site of and otherwise affecting a proposed dam of unprecedented height with the corresponding conditions involved in what is assumed to be the precedent. In the cases of rock-fill dams, there are, among other such conditions, the hardness and toughness of the available rock, the general shape of the pieces or fragments into which it breaks under blasting and the proportion of fines upon such breaking. This, however, is not to say that in all cases all conditions for the dam of greater height need be as favorable as those for the precedent.

Such step by step increase in structure heights is taking place in the case of the new form of deck-type, rock-fill dam, with its deck solely of asphaltic concrete. Thus, in view of the precedent, the 113-ft. Montgomery dam, in Colorado, the authors have developed plans for such a dam which is to have a height of 235 ft.—still low compared with the highest successful rock-fill dams. Construction is being started on this, the Smith Mountain dam for a hydroelectric development on Roanoke River in Virginia. Meanwhile the authors are involved in the consideration of a like dam to be probably somewhat higher than 300 ft., which too would be in the eastern part of the United States.

Greater Economy

Increasingly, rock-fill dams have become a principal means for making economically feasible projects which otherwise would not be so.

In turn, construction and economy of rock-fill dams have been receiving impetus due to the efficiency and economy of equipment for loading and transporting loose rock. It is in large part the manufacturers of such equipment to

whom the engineer must look for further improvement and economies. Because of the large quantities of rock required, even small improvements, reducing unit costs, will be of importance.

The industrial law, almost an axiom, that mass production makes for lower cost applies as well to rock-fill dams. The authors are of the view that the deck-type dam, when utilizing for the deck a material other than earth and hence with the body comprised of rock fill throughout, is likely to be of lower over-all cost than if a considerable part of the cross-section were to consist of another material, such as earth, posing its own problems.

Obviously that view would not be well-founded if the cost of the deck, of material other than earth and therefore comprising an inconsiderable part of the total cross-section, were to be so great as to offset the saving in other respects. However, such appears not to be true, even in the case of a p.c. concrete dam constructed upon a base of individually, and hence rather expensively, placed rock. All the more is it not true of a deck of asphaltic concrete. That material was found to be substantially cheaper than any other for the deck of the Montgomery rock-fill dam. It is believed that this would apply elsewhere.

At Montgomery dam the cost of the asphaltic concrete deck, including all work above or upstream from the rough face of the rock fill, was in 1956-7 less than $14.00 per square yard. The asphaltic deck itself, approx. 12 in. thick, placed in four layers and involving about 27,800 square yards, was constructed in about two months.

Much like the case regarding equipment used for handling rock fill, the great amount of paving required for highways and airfields has encouraged manufacturers to develop asphaltic-concrete, paving equipment which with great efficiency can pave continuously large areas in short time. Adaptation thereof to the placing of asphaltic concrete on a slope has already been accomplished in several cases, though probably not yet in the most efficient form.

Likewise there have been developed mixing plants which, practically entirely on wheels, can readily be transported along highways and set up or removed in a short time; there are large enough for deck construction for rock-fill dams. It is believed that the further adaptation of such paving and mixing equipment will result in still greater economy.

Appendix

Listing and Statistics of Rock-Fill Dams

In the course of their studies of rock-fill dams, the authors have accumulated a mass of information on the subject. Part thereof appears in the tabulations below.

These tabulations were intended to list, with certain standard data, all dams which, in accordance with the proposed definition (see Section A of this paper), appear properly to be classed as rock-fill dams. Application of that definition has resulted in elimination of some dams which heretofore have been referred to by others as "rock-fill" dams. Among the more important of the eliminations are some Algerian dams of individually placed rock or rubble masonry and a few others, the central earth-cores of which appear to exceed, as to cross-sectional area, the rock-fill portions of the dams.

There are two tabulations, the first comprising rock-fill dams in the United States, the second those in foreign countries.

Explanatory

The following is in explanation of the tabulations:

Names of dams are those presently used by the owners. In former years some were known by another name.*

Streams and dam locations, by State or country, are shown. Abbreviations, such as S.F., M.F., etc., preceding the name of a stream, indicate South Fork, Middle Fork, etc., of that stream.

Dams under construction are indicated by "u.c." In some cases, the construction of dams so designated may actually have been completed.

Proposed dams have been omitted, because published data are generally indefinite and in some cases are lacking entirely. Moreover, in the case of any dam merely in the study stage, there is the possibility of substitution of a different type of dam. In this respect, due to advanced stages of planning not known to the authors, some of the omissions may not be justified.

Dams which have failed have not been listed as such. Many, if not most, failures appear to have been due to inadequate spillway provision and not to type of dam or other design features.

Slopes of upstream and downstream faces are shown. The respective figures indicate the ratios which the horizontal component of a given slope bears to the vertical component.

Where different slopes occur on the same face, such are listed in sequence from bottom to top of dam, separated by commas, without details as to elevation at which the change occurs.

Under the caption "Slopes—Interior," applicable to earth cores and earth decks, are listed the slopes of the surfaces of separation between the earth and the major rock fill or fills. In the cases of central earth cores, the upstream earth slopes are stated first, then, following the semi-colon, the downstream slopes.

For central core-walls (other than earth) the sides of which are vertical or nearly so, the interior slopes are indicated with a single figure, "0.0."

Berms have been disregarded, except as affecting the average slopes.

Height of dam is aimed to be the height in feet of the cross-section, at the maximum section of dam, as defined in Section A. This has not always been possible, due to absence of cross-sections in the available descriptions; in such cases the reported height was used.

Type of dam and form of construction are indicated by means of symbols. Capital letters are used for the materials constituting the impervious parts or diaphragms, while small letters are used for other materials, as follows:

A = Asphaltic concrete	c = porous p.c. concrete
C = Portland cement concrete	m = rubble (dry) masonry
E = Rolled earth	r = rock fill
M = Masonry set in p.c. mortar	
S = Steel	
W = Wood	

* Notably, Cogswell (formerly San Gabriel No. 2), Goose Neck (Cheeseman), Hillside (South Lake), Lake Fordyce (Fordyce), Lake Wohlford (Escondido), Mud Mountain (Stevans).

The sequence of the symbols, from left to right, conforms to the sequence of the materials of the dam from upstream to downstream. Thus, by way of example (with the numbers referring to the cross-sections at the end of Section C):

Ar—indicates an asphaltic concrete deck on rock fill (1).
Cmr—indicates a portland cement concrete deck, with rubble masonry, on rock fill (1).
rEr—indicates a central earth-core type of rock-fill dam (4).
Er—indicates an earth-deck type of rock-fill dam (2 and 3).

For p.c. concrete decks, no differentiation is made as between decks of various forms, such as the laminated, sliding or panel forms of construction.

In the cases of dams of the earth-deck type, differentiation between the relatively thin, earth-decks and those which are relatively thick seemed appropriate. Both being indicated by the symbol "Er," such differentiation is effected by the addition of an asterisk where the dam is of the relatively thin, earth-deck type; thus—Er*

Sources of Information

The information contained in the tabulations has been obtained from a great number of engineering periodicals of the United States and foreign countries, engineering hand-books, descriptive publications by Federal agencies, Transactions of International Congresses on Large Dams, etc., etc.

Particularly as to rock-fill dams in foreign countries, the "Statistical Review of Dam Construction," by Robert A. Sutherland, M. ASCE, (Proceedings, ASCE, Vol. 79, Separate No. 355, Nov. 1953) was helpful, though it did not furnish details permitting checking of type. Thus it was not feasible in every instance to determine whether a given dam, designated therein as "rock-fill" or "rock fill-earth fill" dam, is actually a rock-fill dam according to the definition proposed by the authors.

Likewise of great help was the recently published "Register of Dams in the United States," compiled by T. W. Mermel, M. ASCE, for the United States Committee on Large Dams. Taking advantage of its listing of the owners of dams, inquiries for details were addressed by the authors to such owners. On the whole, where inquiries reached them, the cooperation of those owners was excellent; the authors gratefully acknowledge their assistance.

In some instances, as disclosed by data supplied by the owners, designations for dams listed in the Register were found to be incorrect. Corrections in this respect have been incorporated in the pertinent tabulation.

Statement of the sources of information has been omitted, for otherwise the tabulations would have become much longer. However, the authors will be glad, upon request by interested readers, to supply for individual dams the references which were utilized.

Missing Data

As appears from the various blank spaces in the tabulations, the latter are not in every case complete and in some cases may even be in error.

Following each of the two main tabulations is a list of dams for which some of the information was not found. Largely, those dams have been elsewhere reported as "earth fill rock fill" dams; some of them may not properly be classed as rock-fill dams. As to the missing data, the authors appeal and will

be greatful for help from readers. A note or postcard, addressed to one of the authors at 50 Church Street, New York 7, N. Y., will be sufficient and will be individually acknowledged. It should contain a simple sketch of the maximum cross-section of dam, show thereon height, slopes, materials and other information, in general accord with that of the tabulations; preferably also reference to published description. Readers favoring the authors with such information need not be concerned with form or language; that is, the information need not be in English.

The authors hope that as part of the closing discussion they will be able to submit correct and amplified tabulations.

ROCK FILL DAMS IN THE UNITED STATES

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes: Up-stream face	Slopes: Inter-ior	Slopes: Down stream face
ALPINE	Mrm 45	N.F., Stanislaus	Calif. 1906	0.0	-	0.28
ASHTON	Er 60	Snake	Idaho 1917	2.5	0.75	1.25
BARRE FALLS	Er 62	Ware	Mass. u.c.	2.25	1.3	2.00
BEAR CREEK	Er* 223	E.F., Tuckasegee	No.Car. 1954	2.2	1.3	1.30
BEAVER CREEK	Sr 92	Beaver Creek	Colo. ?	1.7	-	1.?
BEAVER PARK	Cmr 87	Beaver Creek	Colo. 1914	0.5	-	1.5,0.5
BELDEN	rEr 150	N.F., Feather	Calif. u.c.	2.5	0.5; 0.1	2.0
BONITA	Cmr 102	Rio Bonito	New Mex. 1931	1.17	-	1.4
BOWMAN (North)	Cmr 175	Canyon Creek	Calif. 1927	0.75,0.50	-	1.4
BROWNLEE	Er* 395	Snake	Idaho u.c.	3.0	1.37	1.4
BUCKS CREEK STORAGE	Cmr 122	Bucks Creek	Calif. 1928	1.4	-	1.5
BUCKHORN	rEr 162	M.F., Kentucky	Kentucky u.c	2.5	0.33; 0.33	2.5
BULL CORRAL	Wmr 240	Huerfano Creek	Colo. 1911	1.0	-	1.25
CAGLES MILL	rEr 148	Mill Creek	Indiana 1953	2.0	0.5; 0.5	2.00
CALISPEL	Cmr 82	N.F., Calispel Cr.	Wash. 1922	?		?
CEDAR CLIFF	Er* 165	E.F., Tuckasegee	No. Car. 1952	?	?	?
CHATWORTH PARK	Cmrm 41	Mormon Creek	Calif. 1892	0.57	-	0.87
CHERRY VALLEY	rEr 330	Cherry	Calif. 1956	2.00	0.70;0.75	2.00
CHRISTIAN VALLEY	Mmr 33	?	Calif. 1916	0.50	-	1.25
CLEAR LAKE	Er 39	Lost	Calif. 1910	3.00	1.00	1.50

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes Up-stream face	Inter-ior	Down stream face
COGSWELL	WCmr 255	W.F., San Gabriel	Calif. 1935	1.35,1.3, 1.2	-	1.50
COURTRIGHT	Cmr 310	Helms Creek	Calif. 1958	1.3,1.2, 1.1,1.0	-	1.30
CRANE VALLEY	rCr 145	N.F., San Joaquin	Calif. 1910	2.0	0.0	1.30
CUCHARAS	Wcmr 125	Cucharas	Colo. 1911	1.00	-	1.50
DALLAS	Er*	Columbia	Ore.-Wash. 1957	2.3, 2.0	1.75	1.5
DAVIS	rEr 138	Colorado	Ariz.-Nev. 1950	?	?	?
DIX RIVER	Cmr 275	Dix	Kentucky 1925	1.2,1.0	-	1.4,1.0
DREW	Wmr 65	Drew Creek	Oregon 1912	0.50	-	1.5
EAGLE GORGE	rEr 230	Green	Wash. 1957	2.15	?	1.3
EAST CANYON	rASAr 93	East Canyon Creek	Utah 1899	0.67	0.0	2.0
FISH LAKE	Er 50	Fish Lake	Oregon 1956	3.0	1.25	1.25
FOURMILE LAKE	Cmr 25	Fourmile Creek	Oregon 1922	0.50	-	1.25
GLASIER LAKE	Cmr 52	Rock Creek	Montana 1937	1.0	-	1.5
GOOSE NECK	SCmr 210	S.F., South Platte	Colo. 1910	0.5	-	1.3
HILLSIDE	Wmr 80	S.F., Bishop Cr.	Calif. 1904	0.75	-	1.25
HORSESHOE	rEr 194	Verde	Arizona 1946	2.0	0.4;0.3, 0.25	2.0
INLAND	Er 195	Warrior	Alabama 1938	3.00	1.3	2.0;1.3
LAGUNA	Cr 43	Colorado	Ariz.-Calif. 1909	?	-	?
LAKE FORDYCE	Cmr 130	Fordyce Creek	Calif. 1926	1.00	-	1.30
LAKE WOHLFORD	Wmr 76	San Elijo Creek	Calif. 1895	0.5	-	1.25,1.00
LEMOLO #1	Cr 120	North Umpqua	Oregon 1954	1.3	-	1.3
LEROY ANDERSON	rEr 260	Coyote Creek	Calif. 1950	2.5	0.75;0.75	2.5
LEWIS SMITH	Er*	Warrior	Alabama u.c.	2.2	1.1	1.3

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes Up-stream face	Inter-ior	Down stream face
LITTLE HELL CREEK	rEr 33	Little Hell Creek	Colo. 1952	3.0	1.0;1.0	2.0
LOWER BEAR RIVER #1	Cmr 233	Bear	Calif. 1952	1.3	-	1.4
LOWER BEAR RIVER #2	Cmr 125	Bear	Calif. 1952	1.0	-	1.4
MEADOW LAKE	Cmrm 73	N.F., Mokelumne	Calif. 1903	0.75,0.5	-	1.0,0.5
MINIDOKA	Er 86	Snake	Idaho 1906	3.0	1.0	1.5
MONTGOMERY	Ar 113	M.F., South Platte	Colo. 1957	1.7	-	1.4
MORENA	CMmr 167	Cottonwood Creek	Calif. 1912	0.9:0.5	-	1.5
MUD MOUNTAIN	rEr 425	White	Wash. 1948	2.25,1.75	0.33; 0.33	2.25, 1.75
NANTAHALA	Er* 250	Nantahala	No.Car. 1942	2.50,1.4	1.37	1.4
ONEONTA	Er 200	?	Alabama ?	3.0	1.0	2.00
OXBOW	Er* 170	Snake	Idaho u.c.	2.5	1.37	1.4
PARADISE	rEr 140	Little Butte	Calif. 1956	3.0	1.0;1.0	2.0
PENROSE-ROSEMONT	SMmr 100	East Beaver Cr.	Colo. 1932	0.50	-	1.4
PLEASANT VALLEY	Er 63	?	Utah 1927	3.00	0.75	1.5
QUEENS CREEK	Er* 78	Queens Creek	No.Car. 1949	2.0	1.3	1.3
RELIEF	CMmr 140	Relief Creek	Calif. 1910	0.5	-	1.50
RHINEDOLLAR	Cmr 30	?	Calif. 1928	0.75	-	1.25
SABRINA LAKE	Wmr 55	M.F., Bishop Cr.	Calif. 1909	0.75	-	1.25
SADDLE BAG	Wmr 47	Leevining Creek	Calif. 1921	0.75	-	1.25
SALT SPRINGS	Cmr 328	N.F., Mokelumne	Calif. 1931	1.3	-	1.4
SAN GABRIEL #1	Cmr 300	San Gabriel	Calif. 1938	1.4,1.35 1.3	-	1.6,1.5 1.4
SAWMILL LAKE	Cmrm 55	Canyon Creek	Calif. 1910	1.0	-	1.0
SKAGUAY	Smr 70	Beaver Creek	Colo. 1901	0.58	-	1.2
SLY PARK	rEr	Sly Park Creek	Calif. 1955	2.5	1.0;0.8	2.0

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes Up-stream face	Inter-ior	Down-stream face
SMITH	rECr 43	Trinchera Creek	Colo. 1916	1.7	0.0	1.3
SMITH MOUNTAIN	Ar 238	Roanoke	Virginia u.c.	1.9	-	1.4
STRAWBERRY	Cmr 150	S.F., Stanislaus	Calif. 1916	1.2,1.0	-	1.35
SWIFT	Cmr 165	Birch Creek	Montana 1914	1.5,1.2	-	1.33
TENNESSEE CREEK	Er* 185	E.F., Tuckasegee	No.Car. 1955	?	?	?
THOMASTON	rEr 142	Naugatuck	Conn. u.c.	2.0	1.4;0.25	1.5
THREE LAKES	Cmr 30	Bucks Creek	Calif. 1896	1.25	-	1.50
TIOGA LAKE	Wmr 18	Leevining Creek	Calif. 1928	0.75	-	1.4
TRINITY	rEr 537	Trinity	Calif. u.c.	3.25	1.0;0.75	2.5
TUTTLE CREEK	rE 157	Big Blue	Kansas u.c.	3.5	0.25	3.0
UPPER BEAR RIVER	Cmr 75	Bear	Calif. 1929	0.75,0.5	-	1.3,0.5
WATAUGA	rEr 328	Watauga	Tenn. 1949	2.0	0.85;0.85	2.0
WISHON	Cmr 250	N.F., Kings	Calif. 1958	1.3,1.2, 1.1,1.0	-	1.4

DAMS IN THE UNITED STATES
regarding which the authors seek
information to permit inclusion, with
complete data, in the foregoing tabulation

Name of dam	State	Year of completion	Height ft.
Animas	Colorado	1906	88
Big Tooth	Colorado	1929	100
Big Vasquez	Colorado	1937	25
Bonham	Colorado	u.c.	38
Box Elder Creek	Utah	1915	68
Elmer J. Chesbro	California	1955	100
Forest Park	Maryland	1909	87
Guineo	Puerto Rico	1951	126
Little Vasquez	Colorado	1937	24
Martindale (South)	Montana	1937	60
Sylacauga	Alabama	?	75
Thomas Valley	Oregon	1923	44
Uvas	California	u.c.	133

ROCK FILL DAMS IN FOREIGN COUNTRIES

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes: Upstream face	Slopes: Interior	Slopes: Down stream face
ALPE-CAVALLI	Cm 135	Loranco	Italy 1926	0.70	-	1.33
AMBUKLAO	rEr 420	Agno	Philippines 1955	2.0,1.75	0.25; 0.25	
AURSJO	Cmr 123	Aura	Norway 1957	1.38,1.24, 1.10,1.00	-	1.38
BERSIMIS	Er* 200	Bersimis	Canada 1957	2.5,2.0	1.38	1.38
BIA	Cmr [illegible]2	Lufira	Belgian Congo 1950	1.2	-	1.4
CARITAYA	CAcmr 125	Caritaya	Chile 1935	1.5	-	1.5
CHARCAS	Cmr 49	?	Mexico 1933	1.25	-	1.4
COGOTI	Cr 245	Huatulame	Chile 1939	1.6	-	1.8
DERBENDI KHAN	rEr 410	Diyala-Sirwan	Iraq u.c.	2.0,1.75	0.3; 0.3	2.0,1.75
DESROCHES	Er* 200	Desroches	Canada 1957	2.5,2.0	1.38	1.38
DEVERO	AMmr [illegible]	?	Italy 1921	0.7	-	1.33
DIGA DI CODELAGO	MAMrm 64	?	Italy 1893	1.4,0.4	-	1.0
EILDON(New)	rEr 260	Goulburn	Australia 1956	2.5	0.6;0.75	2.0
GENKEL	Ar 140	Genkel	Germany 1952	2.25	-	1.75,1.5
HANABANILLA	rEr 154	Hanabanilla Cr.	Cuba 1957	2.0	0.2;0.2	2.0
HARSPRANGET	rECEr 165	Stora Lulealv	Sweden 1951	1.5	0.0	1.5,1.0
HENNE	Ar 200	Henne	Germany 1954	2.25	-	1.75,1.5
HIRFANLI	Er* 282	Kizil Irmak	Turkey u.c.	?	?	?
IRIL-EMDA	CAcr 246	Oued Agrioun	Algeria 1954	1.6	-	?
JYLHAMA	rEWEr 48	Oulujoki	Finland 1949	2.5	0.0	2.5

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes: Up-stream face	Slopes: Interior	Slopes: Down stream face
KAJAKAI	rEr 328	Helmand	Afghanistan 1954	2.5	0.4;0.0	2.0
KARACHUNOUSKIA	Cmr 73	?	Russia ?	1.25,1.1, 1.0	-	1.5
KENNY	Er* 324	Nechako	Canada 1952	2.5	1.37	1.75
LASELE	rEr 75	Angerman	Sweden 1957	?	?	?
LAUGHING JACK	Er* 40	Powers Cr.	Tasmania 1957	1.33	1.33	1.33
MADERO	Cr 153	?	Mexico 1938	1.20	-	1.4
MALPASO	CMmr 255	Mantaro	Peru 1936	0.50	-	1.5,1.33
MIDSKOGFORSEN	rECr 92	?	Sweden 1944	2.0	0.0	1.5
MULUNGUSHI	Ar 117	Mulungushi	U.of S.Africa ?	?	-	?
OSBU	Cmr ?	Aura	Norway 1957	?	-	?
OUED-KEBIR	rCmr 115	Oued-Kebir	Tunisia 1925	1.5,1.0	0.0	1.5,1.0
PARADELLA	Cr 367	Cavado	Portugal 1956	?	?	?
PEGO DO ALTAR	SWAmr 213	Sado	Portugal 1949	1.25	-	?
PINZANES	Cmr 220	?	Mexico 1956	1.2	-	1.3
PRINS	Cmr 110	?	U.of S.Africa ?	1.25	-	1.5
QUIROZ	rEr 164	Chipillico	Peru u.c.	2.5	?	2.5
QUOICH	Cmr 110	Loch Quoich	Scotland 1956	1.3	-	1.4
SAN ILDEFONSO	Cr 203	?	Mexico 1952	?	?	?
SANTO TOMAS	rEr 171	Ixtapandel Oro	Mexico 1946	1.75	?	1.41
SILLRE	Er* 26	?	Sweden 1933	2.0	1.6	1.5
SILVERWOOD	rECr 80	Rosenthal Cr.	Australia 1928	1.5	0.0	2.0
TAXHIMAY	Cmr 134	?	Mexico 1934	0.75	-	1.4

Name of dam	Type and height in ft.	Name of river (creek where stated)	Location and year of completion	Slopes Up-stream face	Inter-ior	Down stream face
TORPSHAMMAR	rECr 73	?	Sweden 1943	2.0,1.5	0.0	1.5
TORRON	Wmr ?	?	Sweden 1936	1.0	-	1.5
VOLTA	Er* 310	Volta	Gold Coast,Africa u.c.	?	?	?
VRLA II	CAmr 82	Vrla	Yugoslavia 1955	1.4	-	1.5
WAYATINAH-B	Er* 65	Derwent	Tasmania 1957	1.33	1.33	1.33

DAMS IN FOREIGN COUNTRIES
regarding which the authors seek
information to permit inclusion, with
complete data, in the foregoing tabulation.

Name of dam	Country	Year of completion	Height ft.
Arghandab	Afghanistan	?	?
Carmel (J. del)	Chile	1929	246
Codelago	Italy	1922	120
Cooby Creek	Australia	1941	?
Dissueri	Italy	1951	135
Excame	Mexico	1951	126
Hunyani Poort	So. Rhodesia	1952	?
Irrigation	Argentina	1934	164
Ishibuchi	Japan	1953	174
Kalk Fontein	U. of So. Africa	?	?
La Joie	Canada	1955	285
Lac Casse	Canada	1955	200
La Rioia	Argentina	1946	141
Le Marinel	Belgian Congo	?	?
Marmore	Italy	?	138
Miboro	Japan	1957	427
Manuherika	New Zealand	1935	110
Melton	Australia	1916	?
Negro	Chile	1930	148
Nozori	Japan	?	210

Name of dam	Country	Year of completion	Height ft.
Pachica	Chile	1929	207
Pend Oreille	Canada	1932	150
Pian Palu	Italy	1955	164
Pintanane	Chile	1929	197
Platani	Italy	1955	217
Pozillo	Italy	1957	179
Rio Tercero	Argentina	1931	177
Rocky Valley	Australia	1957	100
Rust Der Winter	U. of So. Africa	?	105
Sanalona	Mexico	1947	213
Upper Yarra	Australia	u.c.	293
Valsequille	Mexico	1948	269
Verdiana	Italy	1939	105
Vilcuya	Chile	1930	280

Paper 1740 August, 1958 PO 4

Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: THE BERSIMIS SLOPING CORE DAMS[a]

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(Proc. Paper 1740)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

INTRODUCTION

The Canadian Shield area of the Province of Quebec offers some of the world's most favourable sites for development of hydro-electric power. These hydroelectric projects usually involve the construction of large dams. In the past many of the major dams were constructed of concrete, but now, because of modern development in dam construction and also for economic reasons, greater consideration is being given to earth and rockfill dams.

The recently completed Bersimis No. 1, previously known as the Bersimis - Lac Cassé Development (Rousseau, 1956), involved the construction of two earthfill dams and two rockfill dams, which are among the first of their type

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1740 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at meeting of ASCE, Portland, Ore., June 1958.

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to be built in this region. It is proposed to describe in this paper the site and the design, construction and performance of these two rockfill dams.

Topography

The topography of a large part of the Province of Quebec to the north of the St. Lawrence River is rugged and is traversed by several large rivers which empty into the St. Lawrence. One of these rivers, the Bersimis or Betsiamites, flows in a southeasterly direction and reaches the St. Lawrence about 320 miles downstream from the city of Montreal. Bersimis No. 1 is located at the outlet of Lac Cassé approximately 85 miles from the mouth of the Bersimis River. The general location is shown on Fig. 1.

Between the outlet of Lac Cassé and the mouth of the river, the Bersimis, in its natural state, fell approximately 1,150 feet. More than 700 feet of this drop occurred in the first 20 miles below Lac Cassé. This topographic feature offered ideal conditions for the development of power on the river. Conditions also were favourable for construction of a dam at the outlet of Lac Cassé to provide additional head for power development.

At the outlet of Lac Cassé the Bersimis enters a relatively narrow valley where a fall of 90 feet occurs. A small tributary, the Desroches River, joins the Bersimis just downstream of the falls. There appeared to be a choice of constructing dams above the falls across both the Bersimis and Desroches valleys or, alternatively, one dam downstream of the falls and below the confluence of the two rivers. The former alternative was chosen. Fig. 2 shows the location of these dams relative to the locations of the other works included in the project.

Geology

General Geology

The Bersimis dams are located near the southerly boundary of the region known as the Canadian Shield which covers almost one-half the surface area of Canada. This region is an uplifted peneplain of small relief and of elevation almost entirely below 2,000 feet. The Canadian Shield consists of Precambrian rocks which were worn down and rounded off by intensive continental glaciations during the Pleistocene Epoch. These several glaciations completely rearranged the existing drainage pattern and covered the area with an assortment of glacial till and glacio-fluvial deposits. Subsequent to the last glaciation a new and disorganized drainage pattern was created and some resorting of the glacial deposits has occurred through fluvial action. Recent uplift of the land mass has resulted in the existence of marine deposits in the areas immediately adjacent to the lower St. Lawrence River (Stockwell et al, 1957, and Dresser and Denis, 1944).

The rocks of the Bersimis area belong to that part of the Canadian Shield known as the Grenville Series. This is a complex of sedimentary gneisses which have been intruded by anorthosites, gabbros, granites and diorites and their gneissic derivatives, and pegmatites. In the Labrieville area the older sedimentary gneiss or paragneiss and the granitic gneisses are underlain by a batholithic formation of anorthosites and gabbros which outcrops at the surface as intrusions of varying size. A considerable amount of faulting and

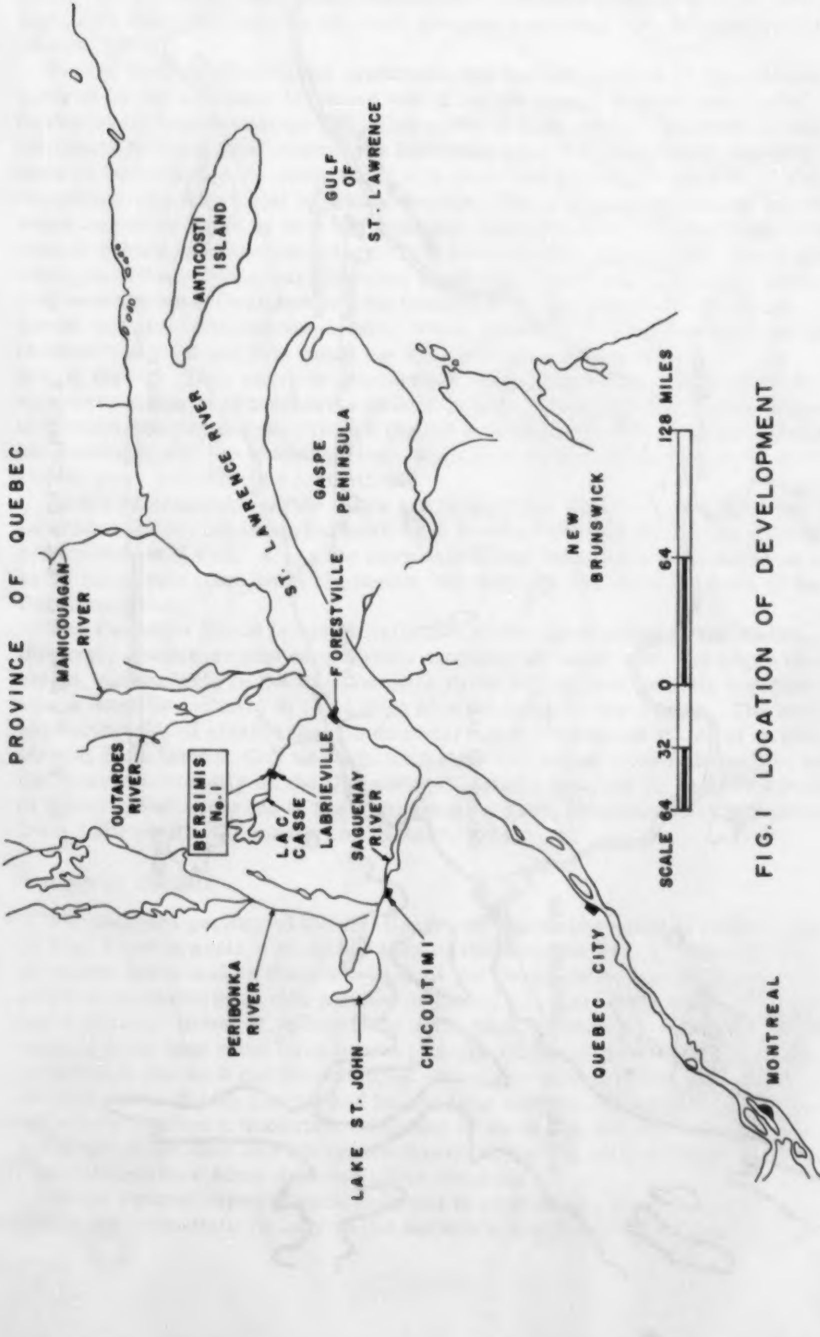


FIG. I LOCATION OF DEVELOPMENT

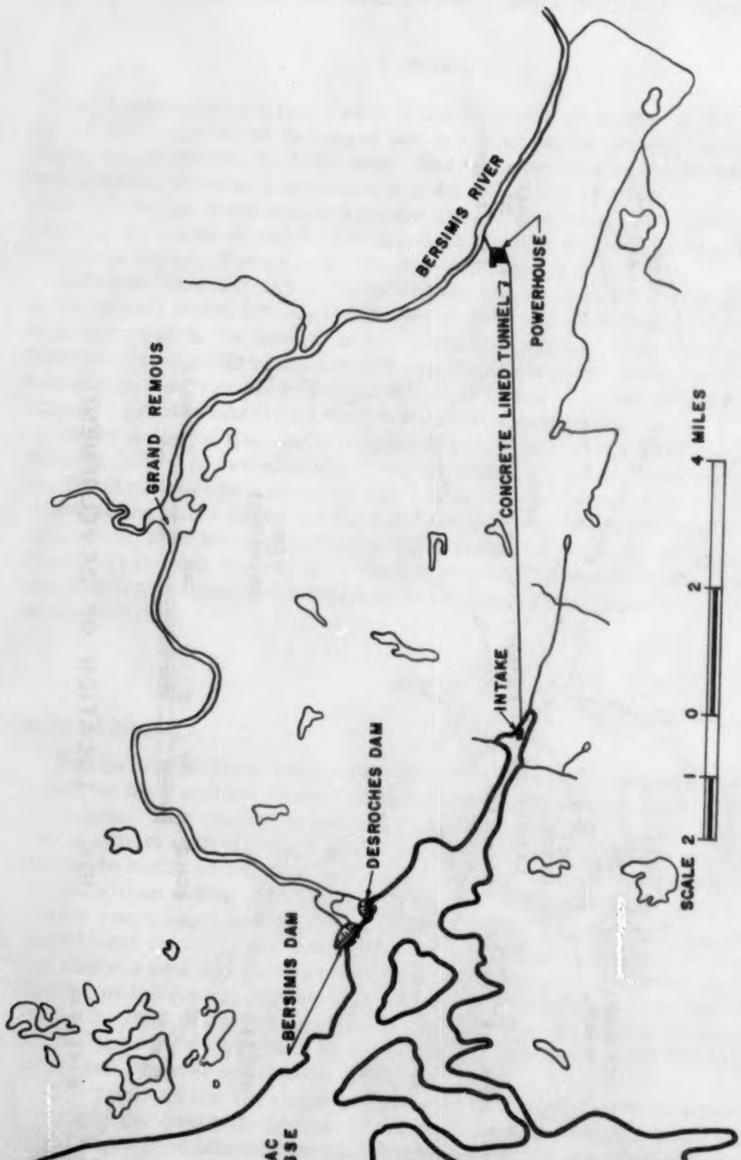


FIG. 2 PLAN OF DEVELOPMENT

shearing has occurred in both Precambrian and Postcambrian times but these fractures are almost invariably well-healed and recemented now. Folding is locally very pronounced in the gneisses and numerous granite, diorite, and pegmatite dykes cut through the rock complex overlying the anorthositic mass (Morin, 1956).

During the last Pleistocene glaciation, the eastern portion of Canada was covered by the Labrador ice sheet which had its centre located near Lake Kaniapiskau, approximately 250 miles north of Labrieville. Because of this proximity to the glacial centre, the Bersimis area was completely scoured down to bedrock and the soils which now exist are entirely the result of glacial deposition or post-glacial erosional forces. The glacial soils consist of tills which are to be found as side hill deposits, and because of the nearness to the glacier centre and the mineralogy of the bedrock they contain only small percentages of fine-grained or clay-type particles. The most commonly occurring overburden formations are the terrace deposits which are of glacio-fluvial and glacio-lacustrine origin. These terraces consist for the most part of stratified silts and fine sands but also include some coarser sand and gravel layers. They occur in thicknesses ranging up to several hundred feet and cover areas of considerable lateral extent. Through these terraces, and to a much smaller degree through the till deposits, the new drainage system has been cut, and the eroded terrace soils have been redeposited as ill sorted recent river and lake bed formations.

Areas of sensitive marine clays are to be found along the shores of the St. Lawrence River, but these have not been found at greater elevations than approximately 400 feet. Along the Bersimis River these have been noted as far as 45 miles upstream from the mouth, but they are absent in the area of the Bersimis dams.

The Canadian Shield is seismically one of the least active zones known. However, shallow shocks of moderate intensity do occur near the edges of the shield, particularly in the St. Lawrence River valley, and seismic considerations must be included in the design of structures in this region. The nearest earthquake of sizable magnitude occurred in 1925 about 70 miles downstream from Quebec City and approximately 125 miles from Labrieville, and its recorded intensity on the Gutenberg-Richter scale was 7. In 1935 a tremor of intensity 6.2 occurred in the Temiskaming area, approximately 500 miles from Labrieville (Gutenberg and Richter, 1954).

Geology of the Site

The detailed geology of the Bersimis and Desroches dams is shown in plan on Fig. 3 and in section along the axes of the dams on Fig. 4. Beneath the Bersimis River and on the south bank of the Bersimis valley, bedrock outcrops or is overlain by thin patches of glacial till and lenses of sand, gravel, and boulders. However, beneath the north bank the bedrock surface dips downward to form what must have been a pre-glacial drainage channel. This depression in the rock has been infilled with a succession of till and varved clay and silt strata during glacial and interglacial periods. This complex of glacial materials reaches a maximum thickness of about 300 feet beneath the extreme north end of the dam and decreases thereafter as the rock surface rises to reach the surface some distance above the dam.

Of the several types of rock referred to previously, the only ones found within the immediate vicinity of the Bersimis and Desroches dams are the

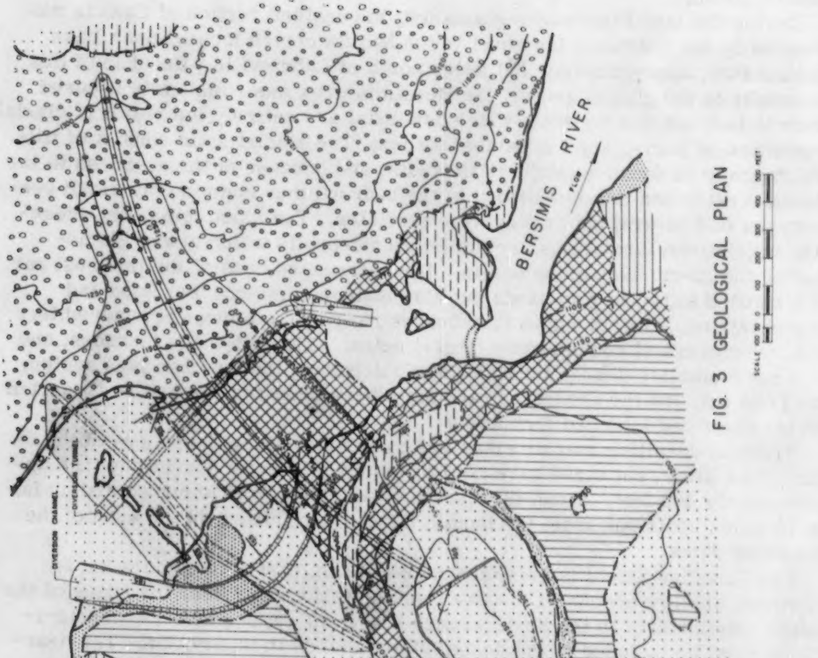


FIG. 3 GEOLOGICAL PLAN

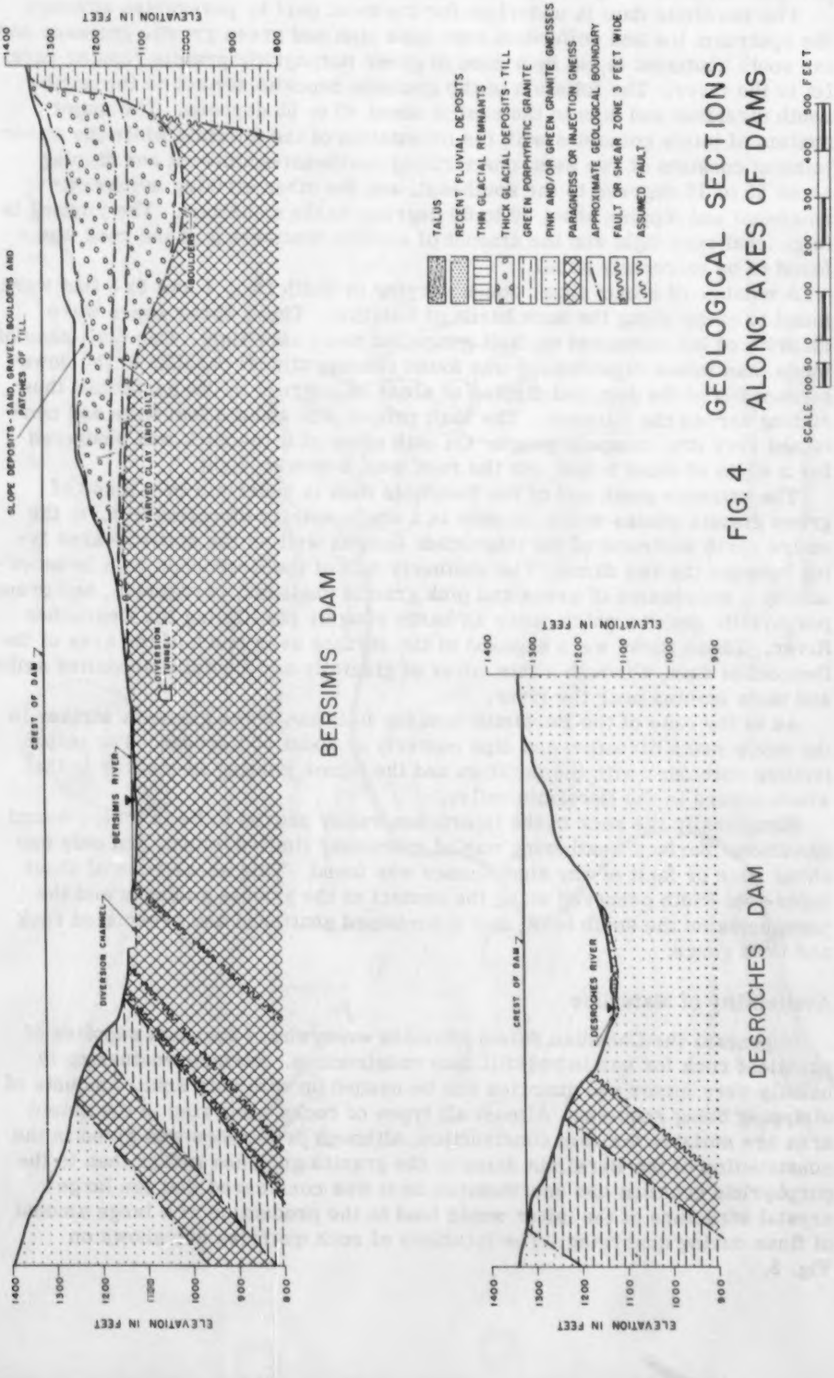


FIG. 4 GEOLOGICAL SECTIONS ALONG AXES OF DAMS

paragneiss, pink and green granite gneisses, and green porphyritic granite.

The Bersimis dam is underlain for the most part by paragneiss although the upstream toe and cofferdam rest upon pink and green granite gneisses and the south abutment is cut by a zone of green porphyritic granite running parallel to the river. The foliation of the gneissic deposits strikes in the north-south direction and dips to the east at about 45 to 65 degrees. The major system of joints coincides with the orientation of the foliation while the minor jointing consists of two sets, one striking northeast-southwest and dipping about 55 to 75 degrees to the southeast, and the other striking northwest-southeast and dipping about 50 to 60 degrees to the southwest. The jointing is in general very tight and the amount of surface weathering of the rock was found to be extremely small.

A number of minor shear zones varying in width from 1 foot to 3 feet were found to occur along the dark bands of foliation. These shear zones were chloritized but contained no fault gouge and were very tight. One fault zone of more than minor significance was found running almost parallel to the downstream toe of the dam and dipping at about 85 degrees to the northeast, thus cutting across the foliation. The fault proper was about 3 feet wide and contained very dry, compact gouge. On both sides of it the rock was shattered for a width of about 5 feet, but the rock was, however, tight.

The extreme south end of the Bersimis dam is underlain by a mass of green granite gneiss which extends in a southeasterly direction to form the entire north abutment of the Desroches dam as well as the spillway area lying between the two dams. The southerly half of the Desroches dam is underlain by a succession of green and pink granite gneisses, paragneiss, and green porphyritic granite which occur as bands roughly paralleling the Desroches River. These rocks were exposed at the surface over much of the area of the Desroches dam, although a thin cover of glacially and fluvially deposited soils and talus existed near the river.

As in the case of the Bersimis dam the foliation of the gneisses strikes in the north-south direction and dips easterly at about 55 degrees. The major jointing coincides with the foliation and the minor jointing is similar to that which occurs in the Bersimis valley.

Structurally the rock in the Desroches valley proved to be in a very sound condition. Surface weathering was of extremely limited extent, and only one shear zone or fault of any significance was found. This shear zone of about three-foot width occurred along the contact of the granite gneisses and the paragneiss on the south bank, and it contained shattered and brecciated rock and fault gouge.

Availability of Materials

In general the Canadian Shield provides everywhere abundant supplies of excellent rock for use in rockfill dam construction. Surface weathering is usually very minor and quarries can be opened up with only small amounts of stripping being required. Almost all types of rocks occurring in the shield area are suitable for dam construction, although preference was given in the construction of the Bersimis dams to the granite gneisses rather than to the porphyritic granites and anorthosites as it was considered that the large crystal structure of the latter would lead to the production of a large amount of fines during quarrying. The locations of rock quarries are shown on Fig. 5.

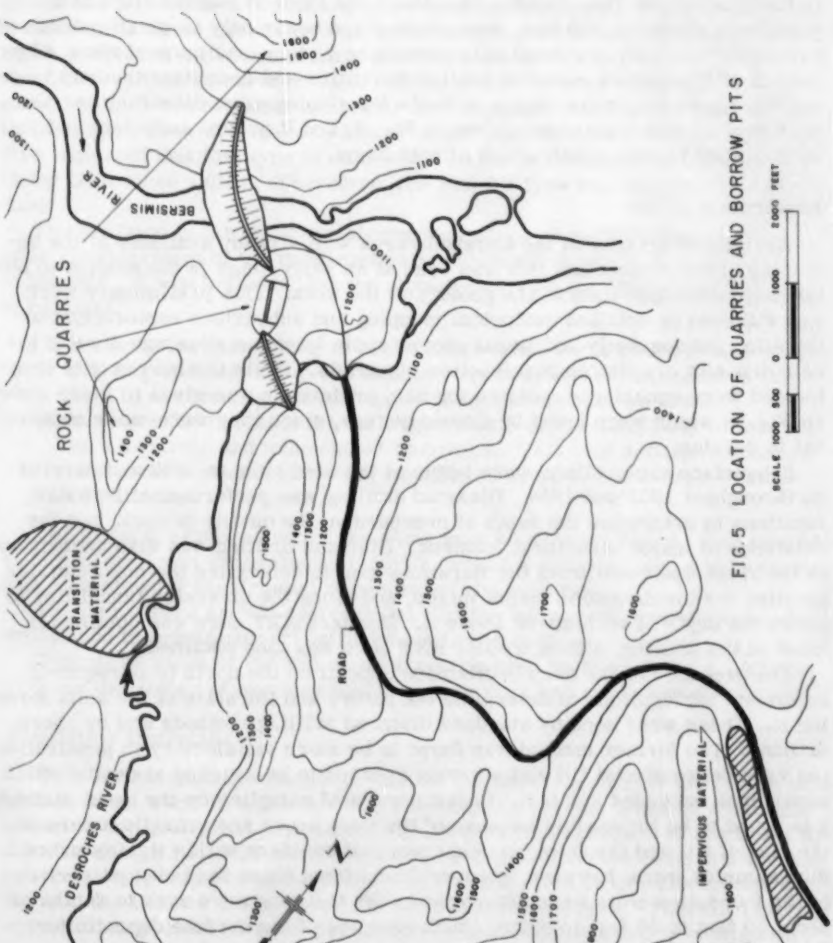


FIG 5 LOCATION OF QUARRIES AND BORROW PITS

Material for transitions is usually available within economic hauling distances from glacio-fluvial or recent fluvial deposits. Gravel deposits are usually fairly well graded and reasonably clean, but sand deposits, particularly when they occur as terrace formations, tend to be uniformly graded and on the fine side. For the Bersimis dams the transition materials were obtained from a gravel deposit located as shown on Fig. 5.

Materials suitable for use in impervious cores do not occur in abundance in this part of the Canadian Shield. Above the limit of marine clays at approximate elevation 400 feet, water-lain clays occur only as small patches of varved clays. Clayey glacial tills containing up to a maximum of about 10 per cent of clay particles occur in limited quantities and constitute the only impervious material in the region suitable for sloping-core construction. One such deposit was located as shown on Fig. 5, and it proved sufficient and satisfactory for the construction of both dams.

Exploration of Site

Aerial photography of the Bersimis area was already available at the beginning of the project and this was used at an early stage in the studies to obtain a general appraisal of the geology of the area. This preliminary work was followed by detailed geological mapping and subsurface explorations at the site. Subsequently additional photographic interpretation was done to locate deposits of suitable construction materials. While the borrow pits thus located were considered suitable for use, preference was given to those shown on Fig. 5, which were found by ground survey, since they were more economical to develop.

Subsurface explorations were begun at the dam sites in 1952 and carried on throughout 1953 and 1954. Diamond drilling was performed at both dam locations to determine the depth of overburden, the quality of rock, and the existence of major structural features. Diamond drilling was also undertaken in the river upstream from the Bersimis dam to determine the most suitable location for the diversion tunnel intake, and along the diversion tunnel line to prove the depth of rock cover above it. Standard AXT core was obtained in most of the drilling, although some EXT core was also obtained.

The presence of the deep overburden deposit on the north bank required additional explorations to determine the nature and the state of the soils forming it. These were done by standard diamond drilling methods and by churn drilling. The former method was found to be more satisfactory in penetrating the very dense glacial till and allowing bedrock to be reached at depths which sometimes exceeded 250 feet. Undisturbed soil sampling by the usual methods was found to be impossible because of the very dense and gravelly nature of the glacial till and the frequent occurrence of boulders within it. Disturbed tube samples were, however, recovered and from these the index properties of the foundation soils were determined. Six test pits were sunk to depths of about 50 feet to 60 feet to obtain chunk samples of the surface deposits for laboratory testing.

Geophysical methods were employed in the early stages of the explorations to determine the elevation of rock along two profiles through the north bank of the Bersimis River. The seismic method was used and, although it did produce profiles that were in general agreement with those subsequently determined by drilling, it was in error by as much as 45 feet in a 220-foot depth. For the most part the geophysically determined rock surface was higher than the actual one found by drilling.

Design

General Arrangement

The site chosen for the dams at Lac Cassé required the construction of a dam in the Desroches valley 1,100 feet long and 225 feet high, and a dam in the Bersimis valley 2,100 feet long and 200 feet high. The foundations for the Desroches dam were of sound rock and, therefore, this site provided ideal conditions for construction of any of several types of dam. The north bank of the Bersimis River, however, was covered with a thick mantle of overburden which precluded the consideration of a mass concrete dam for that site. It was believed that an earth or rockfill dam would be most suitable for construction on the overburden and, after investigation of the availability of materials, a decision was made in favour of the rockfill type. Economic studies also indicated that this type of dam would be the best choice for construction in the Desroches valley. Therefore, the rockfill type was adopted for both dams.

The decision in favour of the sloping-core type of rockfill dam was made after consideration of the construction methods which would be most adaptable at the site. It was believed that the sloping-core design offered the following advantages:

(1) It allows for the placing of rock in high lifts. As a result a minimum of building and rooting up of roads is required, and rock limited in size only by the loading and transporting equipment may be used.
(2) It allows for the placing of a large volume of the main body of the dam as a separate and independent operation. This is a definite advantage where core construction may be delayed because of adverse weather conditions.

The general arrangement of the dams may be seen on Fig. 6. The common rock abutment was excavated to form an overflow spillway, and a gate section was subsequently constructed at one end of the spillway.

The dams were arched upstream so that the hydrostatic reservoir load would tend to induce compressive forces in the cores.

Typical sections of the two dams are presented on Figs. 7 and 8. These indicate the design of sections based on both rock and earth foundations.

Foundation Materials

With the exception of the north bank of the Bersimis River, the foundations of both dams were stripped down to bedrock, the various types of which are shown on the geological plan on Fig. 3. In the north abutment of the Bersimis dam the design required the impervious core to be carried down in a trench to a relatively impervious stratum. Over much of its length this trench was cut into the surface of the upper till layer shown on Fig. 4. This till layer is approximately 100 feet in thickness and is underlain by a fairly continuous layer of varved clay and silt of about 25 feet in thickness and a lower till layer which is about 100 feet thick and located beneath the extreme north end of the dam.

Both till layers have similar properties and for design purposes no distinction was made between them. The tills together with the interposed varved clay layer exist naturally in a very dense state, and it is believed that they have been overconsolidated to a moderate degree. The ranges of particle size

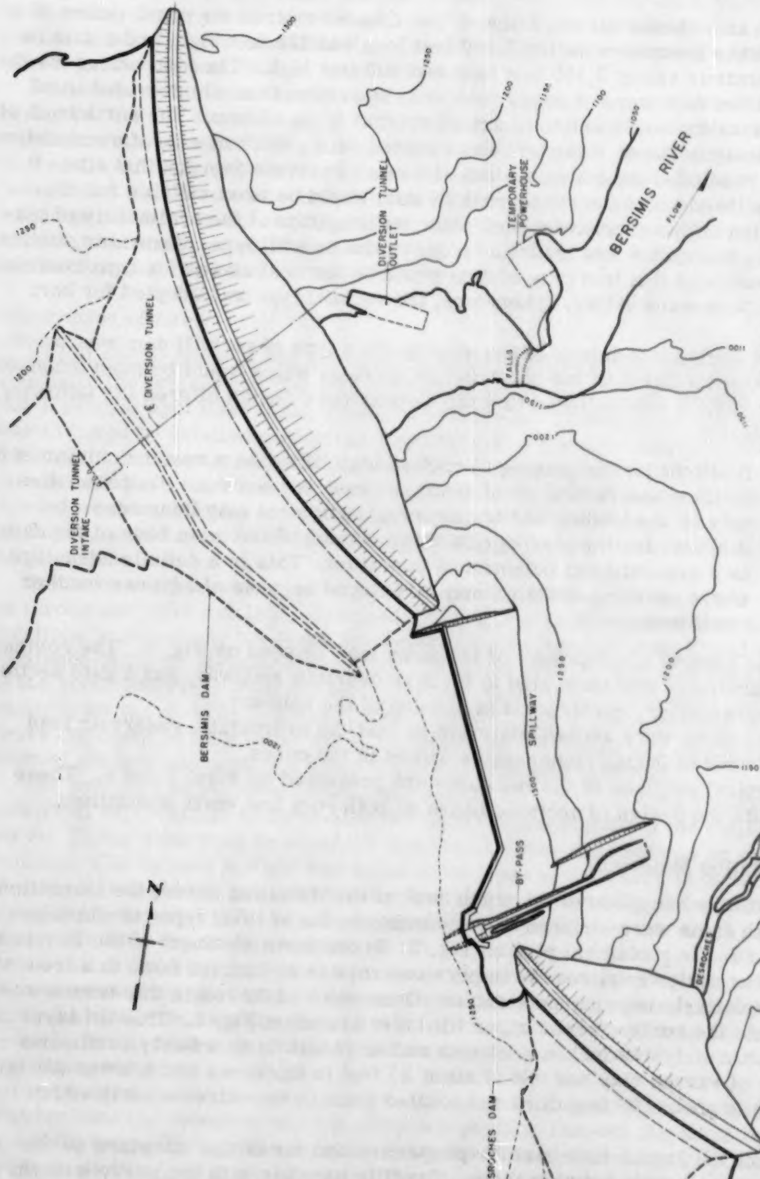


FIG. 6 GENERAL ARRANGEMENT OF DAMS

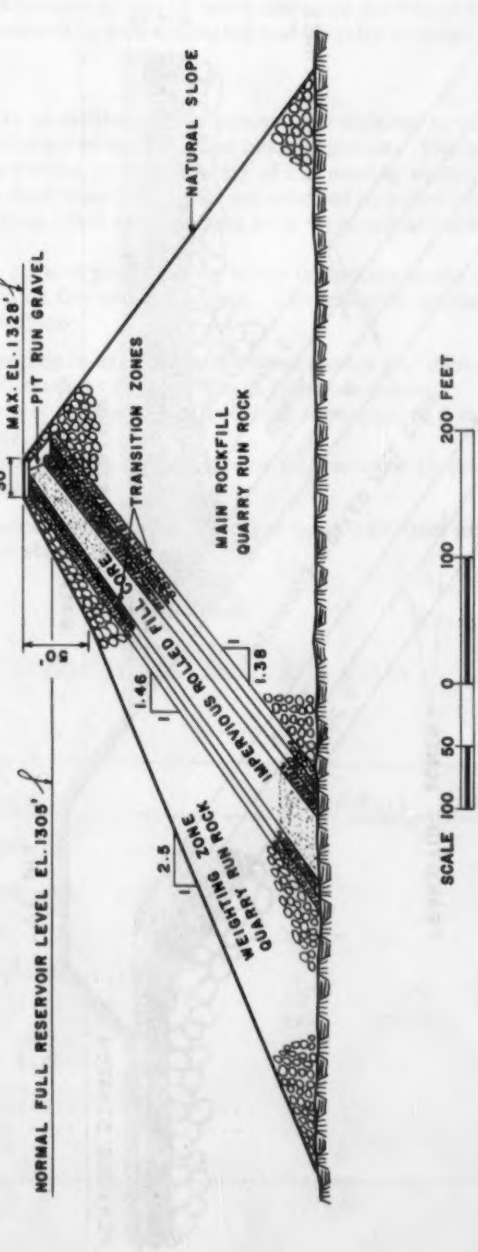


FIG. 7 BERSIMIS AND DESROCHES DAMS — TYPICAL SECTION

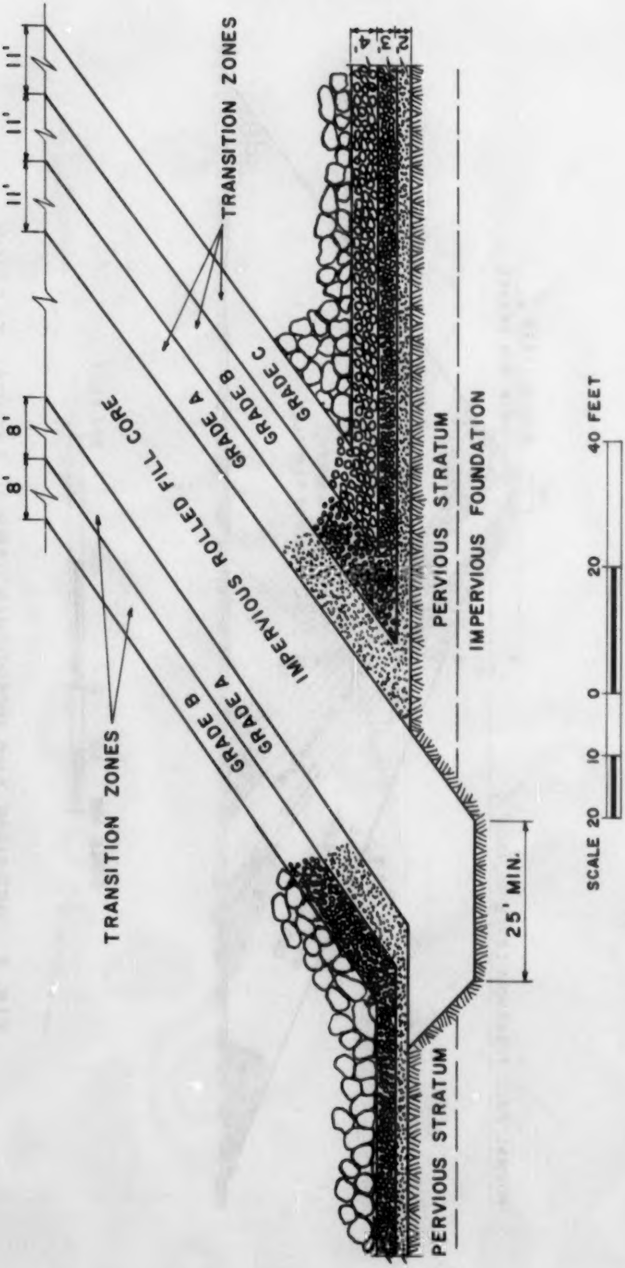


FIG. 8 BERSIMIS DAM – PARTIAL SECTION ON OVERBURDEN

gradations for the two materials are shown on Fig. 9 and the other know soil properties for them are summarized in Table 1. The values of the effective shear strength parameters c' and Ø' were assumed for the purposes of calculation and are considered to be reasonable and slightly conservative.

Dam Materials

Rock for the main rockfills and the upstream weighting zones of both dams consisted almost entirely of sound green granite gneiss. The maximum rock size allowed was governed by the capacity of the moving equipment, while the minimum size was rock dust which was not allowed to constitute more than 15 per cent of any load. Soil and unsound rock were not allowed to be placed in the rockfill.

The impervious core is protected by three transition zones on the downstream side and two on the upstream side. The material gradations that were specified were as follows:

- Grade A - Ranging in size from not more than 2 per cent passing a No. 200 sieve to that passing a 1/4-inch sieve.
- Grade B - Gravel ranging from 1/4-inch minimum to 3 inches maximum size.
- Grade C - Rock ranging from 3 inches minimum to 10 inches maximum size.

Clean gravel was screened and the transition materials thus produced had gradation ranges as shown on Fig. 10.

TABLE 1

Bersimis Dam
Properties for North Bank Soils

Property	Dense Till	Varved Clay and Silt
Specific gravity	2.73	2.73
Natural water content - per cent ..	10 - 20	18 - 25
Natural bulk density - lb/cu ft ...	127 - 145	125 - 134
Coefficient of permeability - cm/sec	10^{-2} to 10^{-5} Avg = 10^{-4}	10^{-6}
Effective shear strength parameters:		
c' - psf	720	720
Ø' - degrees	35	35

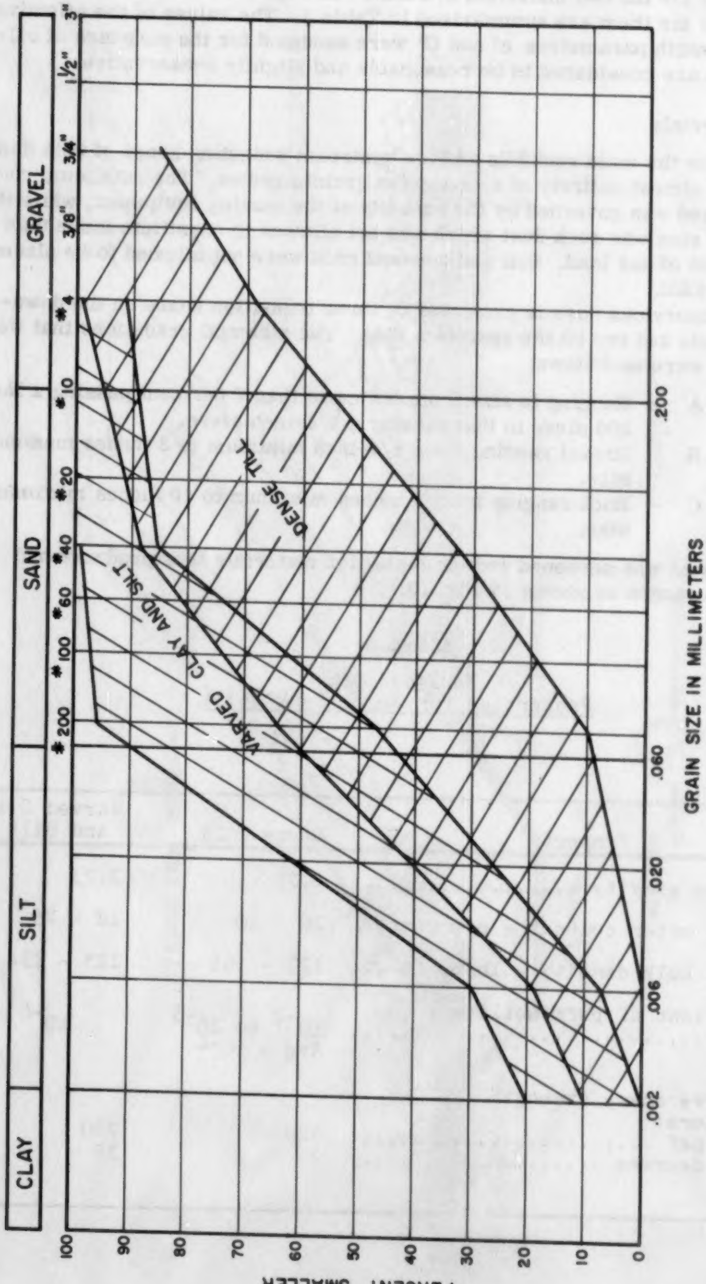


FIG.9 BERSIMIS DAM NORTH ABUTMENT – PARTICLE SIZE DISTRIBUTION OF FOUNDATION MATERIALS

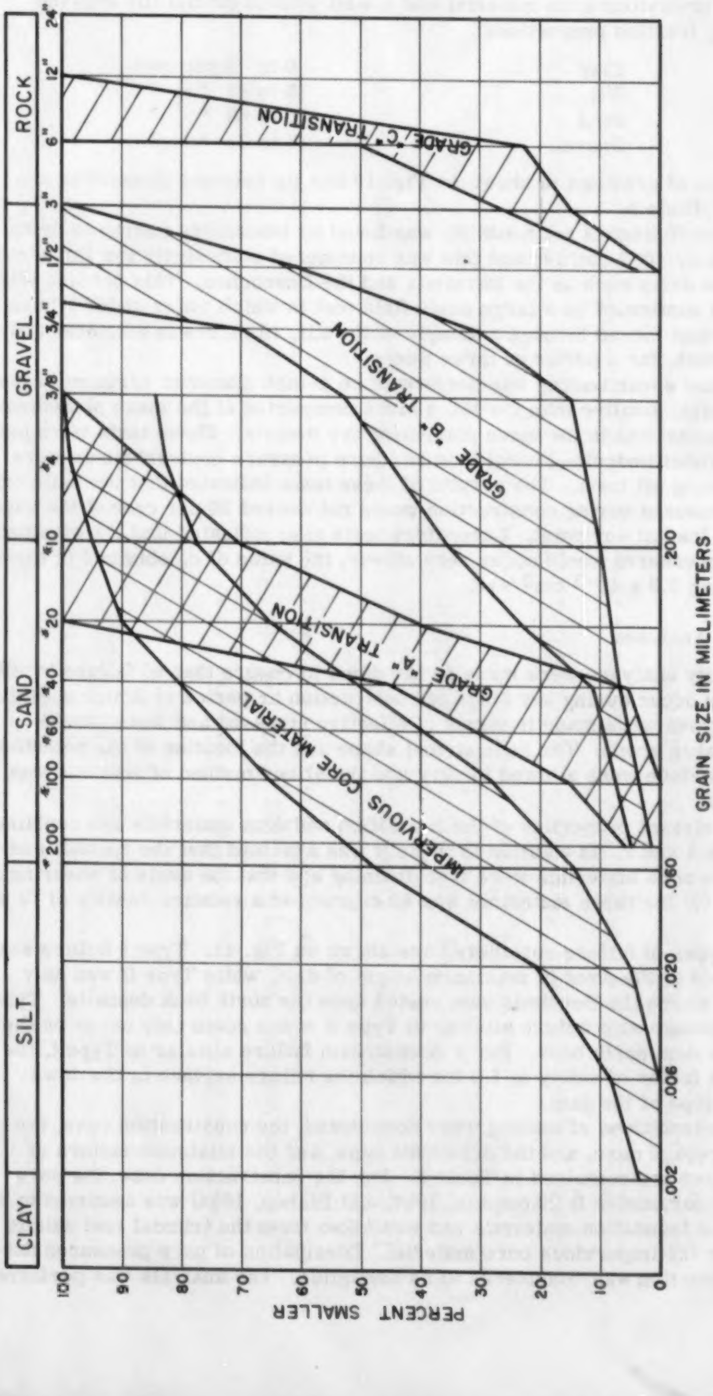


FIG.10 PARTICLE SIZE DISTRIBUTION OF IMPERVIOUS CORE AND TRANSITION MATERIALS

The impervious core material was a well-graded glacial till with the following fraction proportions:

Clay	0 to 5 per cent
Silt	10 to 45 " "
Sand	30 to 80 " "
Gravel	5 to 25 " "

The range of gradings is shown on Fig. 10 and its relevant properties are listed in Table 2.

The coefficient of permeability was found by laboratory testing to be approximately 10^{-6} cm/sec and this was considered sufficiently low for use in core type dams such as the Bersimis and the Desroches. This permeability was also confirmed by a large scale field test in which water under a head of 450 feet was forced through a sample of the till, 18 inches in diameter and 30 inches thick, for a period of three weeks.

Triaxial shear testing was performed on 4-inch diameter samples of the till material smaller than the No. 4 sieve compacted at the mean placement water content and to the mean placement dry density. These tests were performed under undrained conditions and pore pressure measurements were made during all tests. The results of these tests indicated that the build-up of pore pressures during construction would not exceed 20 per cent of the superimposed load at any point. Laboratory tests also indicated that the dissipation of pore pressures would occur very slowly, the value of c_v obtained in these tests being 3.2×10^{-3} cm^2/sec.

Stability Analyses

Stability analyses were made on the dams to ensure that no failure by sliding would occur during any stage of construction or period of future operation. The analyses were made in terms of effective stresses and were two-dimensional in scope. The geometrical shape and the location of the potential failure surface were allowed to vary and the slices method of analysis was employed.

The relevant properties of the foundation and dam materials are contained in Tables 1 and 2. In addition to these it was assumed that the rockfill and the transition zone materials were free draining and that the angle of shearing resistance (ϕ) for these materials was 42 degrees at a relative density of 70 per cent.

The types of failure considered are shown on Fig. 11. Type I failure was considered at the point of maximum height of dam, while Type II was only possible where the Bersimis dam rested upon the north bank deposits. Type III is a downstream failure similar to Type II which could only occur on the Bersimis dam north bank. For a downstream failure similar to Type I, the minimum factor of safety is 1.0 for which the failure surface is the downstream slope of the dam.

Three conditions of loading were considered, the construction case, the steady seepage case, and the drawdown case, and the minimum factors of safety found are contained in Table 3. For the construction case, the pore pressure parameter $\bar{B}$ (Skempton, 1954, and Bishop, 1954) was assumed to be 0.4 for the foundation materials and was taken from the triaxial test data to be 0.2 for the impervious core material. Dissipation of pore pressures during construction was considered to be negligible. The analysis was performed

TABLE 2

Bersimis Dams

Properties of Impervious Core Material

Specific gravity	2.73
Optimum water content - per cent - (Fraction $<$ No. 4 sieve)	9.3
Mean placement water content - per cent - (Fraction $<$ No. 4 sieve)	9.6 = 1.03 x Optimum
Maximum dry density by standard Proctor compaction test (Corrected for material $>$ No. 4 sieve) - lb/cu ft	135
Mean placement dry density - lb/cu ft	132 = 0.98 x Proctor maximum
Coefficient of permeability - cm/sec	1 x 10^{-6} to 4 x 10^{-6}
Effective shear strength parameters:	
c' - psf - construction case	1,300
c' - psf - steady seepage and drawdown cases	500
ø' - degrees	36

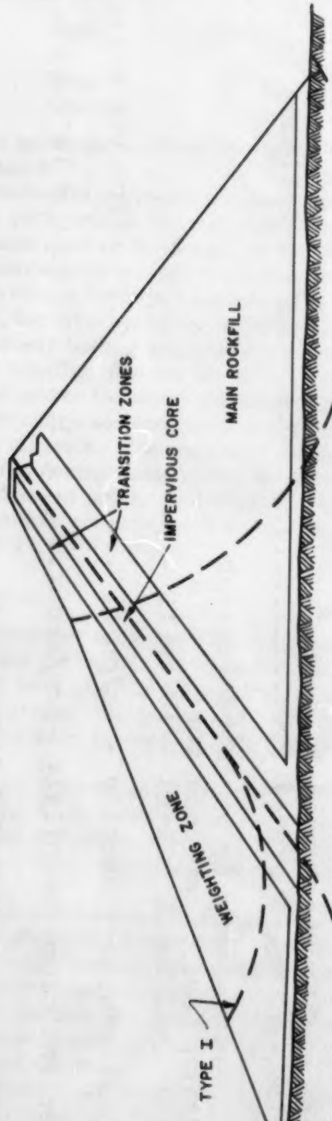


FIG.II TYPES OF FAILURE SURFACES CONSIDERED IN STABILITY ANALYSES

for the condition when the dam was completed and the reservoir empty, and as it was stable for this condition a failure at an intermediate stage in construction was impossible. The analysis was then repeated for varying reservoir levels, and the minimum factor of safety of 1.80 was found to occur with a Type II failure and the reservoir elevation below 1170 feet. The variation of factor of safety with reservoir level is shown on Fig. 12. In reality the reservoir elevation rose during the construction of the dam and the effect of this was to produce actual factors of safety greater than those shown in Table 3.

In the analysis for the steady seepage case, it was considered that the minimum probable operating reservoir level would be 1275 feet and that steady seepage conditions would not be set up with reservoir levels lower than this. Complete saturation of the impervious core was assumed to have occurred when the steady seepage flow net became established. The pore pressures were determined from the latter and a decrease in c' due to saturation of the fill was allowed for. For these conditions the minimum factor of safety of 2.15 was found for a Type I failure with the reservoir elevation at 1275 feet.

For the drawdown case it was assumed that steady seepage conditions had been established before drawdown occurred. Therefore, $\bar{B}$ was assumed to be 1.0. Drawdown to various levels was considered to take place instantaneously and the minimum factor of safety of 1.70 for a Type II failure was found to occur when the reservoir is lowered below elevation 1175. The variation of factor of safety with drawdown elevation is shown on Fig. 12.

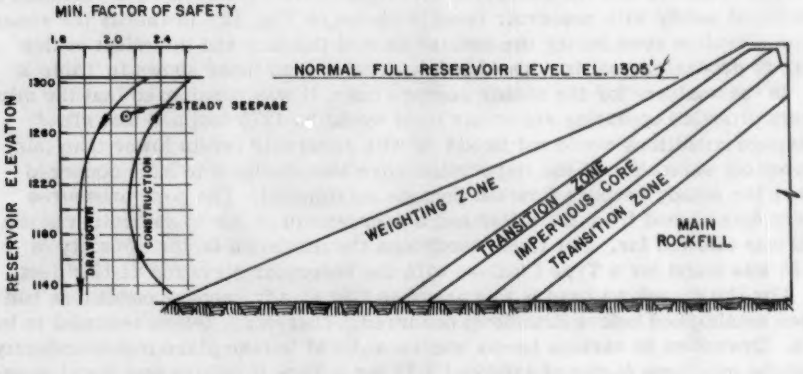
Settlement

In the design of the Bersimis dams it was realized that settlement would occur and that an allowance should be made for it. However, because of the

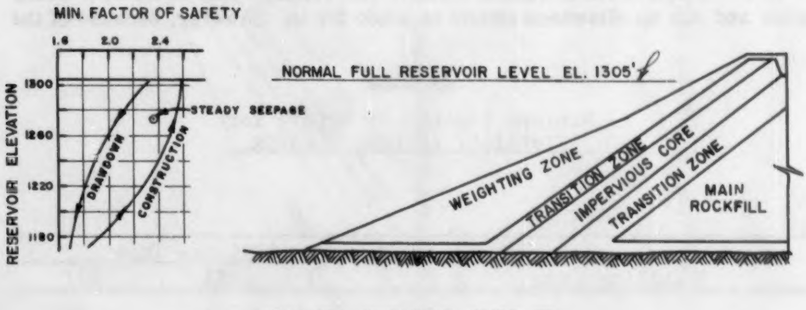
TABLE 3

Minimum Factors of Safety for Stability Against Sliding

Stability Case	Failure Type I	Failure Type II	Failure Type III
Construction .... (Dam completed and reservoir rising)	2.15	1.80	2.55
Steady seepage .... (Failure Type / Reservoir Elevation) (I & II 1275 feet) (III 1312 feet)	2.15	2.35	2.25
Drawdown .... (Minimum values for drawdown to elevation 1175 or lower)	1.75	1.70	Condition Unaffected by Drawdown



FAILURE SURFACE TYPE I



FAILURE SURFACE TYPE II

FIG. 12 VARIATION OF FACTOR OF SAFETY WITH RESERVOIR ELEVATION

sloping-core nature of the dams it was evident that the settlement of the dam crests would result from movements in the main rockfill, and that such settlements cannot be rationally calculated but must be estimated from previous experience. It was assumed that settlement in the rockfill would amount to about 5 per cent of its height and that at least two-thirds of this would occur during construction. To accommodate the movements expected after construction, ample superelevation was included in the design.

Foundation settlement is confined to the north bank deposit of the Bersimis dam, but because of the nature of these soils good samples for consolidation testing were very difficult to obtain and this precluded any accurate settlement calculation. However, a rough estimate, based on limited consolidation test data, showed that the foundation settlement would be very small. Since only a small portion of this would occur after construction, no additional allowance in superelevation was made.

Seepage and Transition Design

Seepage from the reservoir in the immediate vicinity of the Bersimis dams was considered to be possible directly through the core, through the bedrock beneath or around the dams, and through the north bank deposits. Seepage through the core was estimated, on the basis of the coefficient of permeability shown in Table 2, to be of the order of 5×10^{-2} cfs for the Bersimis and 2.5×10^{-2} cfs for the Desroches dam. Seepage through the surrounding rock mass was considered to be of much greater magnitude than seepage through the core, and to reduce this flow of water extensive cement grouting was carried out beneath the impervious core where it rests upon rock. Blanket grouting to a depth of 15 feet was done on a 10-foot square grid spacing over this entire area, and in addition to this a grout curtain was constructed extending into rock from 50 to 60 per cent of the acting head of water. As the surrounding rock abutments were composed of sound rock, no additional abutment grouting beyond the limits of the dams was deemed necessary.

During the design of the Bersimis dam, the possibility that the north bank foundation might prove to be more pervious than was considered desirable became a matter of concern. This concern was increased during excavation of the north bank core trench when some evidence of stratification amongst the more pervious parts of the deposit was uncovered. However, the impervious core was carried down to depths as great as 50 feet, and it was considered that any seepage passing beneath the core would be of a minor nature. As an added precaution, an attempt was made to seal any underlying pervious layers or lenses by bentonite injections. Grout acceptance by the foundation materials was very small.

Transition zones on both the upstream and downstream sides of the impervious core were designed and specified as outlined previously. These gradation specifications satisfy the filter requirements as outlined by Karpoff (1955), and the materials actually produced satisfied these requirements with the exception of the finer portion of Grade B material. To check on the possibility of particle migration with the actual materials a large size permeability test was conducted at the site with an effective hydraulic gradient of 180. After three weeks operation the steel pipe permeameter was cut open and the transition materials were examined, but no evidence of migration was found. On the basis of this test and in view of the fact that the maximum hydraulic gradient in either dam is less than 10, it was concluded that the transitions as

constructed would perform satisfactorily. The thicknesses of the transition zones were determined on the basis of the minimum widths which were considered practicable.

Construction

Schedule and Progress

The schedule required construction to be started in the fall of 1953 and to be completed before the end of 1955. In keeping with this schedule, the contractor set up a camp near the dam sites and began stripping the foundations and quarry areas in late 1953. Grouting of foundations was also started before the end of the year. The cofferdams, diversion tunnel and diversion intake were completed by the spring of 1954.

The rockfill operation was begun in the summer of 1954 and proceeded according to schedule, but the progress on impervious core construction was slow due to unusually wet weather. The 1955 season was much more favourable for the construction of earth fill, and the dams were completed, as scheduled, before the end of the year.

Diversion

The diversion of the Bersimis River flows was complicated by the necessity to supply water to a temporary hydroelectric plant installed at the head of the falls immediately downstream from the dam. A diversion tunnel 39 feet wide by 36 feet high was constructed beneath the river bed, and a rockfilled timber crib weir was constructed at the outlet to maintain a forebay for the temporary power house. A channel was also excavated to divert the river flows until the diversion tunnel was completed. The upstream and downstream cofferdams were constructed of rockfill with impervious blankets.

The diversion intake was provided with two gates 19 feet wide by 34 feet high and capable of operating under a head of 180 feet. These were closed at the time scheduled for filling the reservoir. The intake also contained a 102-inch diameter discharge regulator, which was operated to discharge sufficient water through the diversion tunnel during the reservoir filling period to operate the temporary power plant. After the Bersimis power house was placed in operation in late 1956, the discharge regulator was closed and a concrete plug was placed in the tunnel beneath the impervious core of the dam.

Diversion of the Desroches River flows was effected by constructing a concrete culvert along the bottom of the steep right bank.

Foundation Treatment

With the exception of the north abutment of the Bersimis dam, the design of both dams required that the foundations be stripped to rock. In the case of the Bersimis dam, however, the main body of rockfill was dumped on a boulder field overlying the bedrock in the river bed, and the upstream cofferdam which formed the base of the loading zone was constructed at its north end on relatively impervious overburden in the river bed.

The north bank of the Bersimis River was stripped of topsoil, and the earth abutment was cut to a regular slope to receive the base transition. The cutoff in the north bank was carried deeper than original plans called for due to the occurrence of pervious lenses in the foundation.

Small volumes of rock were excavated in the core foundation area, especially at the Desroches Dam, to correct adverse slopes. Concrete was placed in some areas where uneven rock would make it difficult to compact impervious fill.

Special treatment at the Desroches dam was necessary in a shear zone intersecting the foundation of the impervious core. The gouge was removed to a depth of about 3 feet from the upstream to the downstream limits of the core, and the trench formed was filled with concrete. A shaft was also sunk along the dip of the shear zone to a depth of about 20 feet, and this was filled with concrete to form a cutoff.

Guniting of the rock surface prepared for the impervious core was carried out over a considerable area. The gunite was effective in sealing open joints during the blanket grouting operation.

Quarrying

The two main quarries were located in a hill just upstream of the two dams. During the 1954 construction season a supplementary quarry was operated on the north bank of the Bersimis, using the coyote hole method. In 1955 a quarry was also opened on the spillway site between the two dams. With the exception of the coyote hole operation on the north bank of the Bersimis, all quarrying was carried out by the benching method.

The following is a list of equipment used for quarrying and transporting rock:

<u>Drills</u>

2 Quarrymasters
5 Nine-inch churn drills

<u>Shovels</u>

1 - 5 yard
2 - 3-1/2 yard
4 - 2-1/2 yard

<u>Trucks</u>

14 - 35 ton
12 - 22 ton

Borrow Pits

A screening plant was set up about one-half mile from the source of transition materials, and the gravels were separated into the three grades required for the dams. Since the pit lay in the area to be flooded, the transition materials were screened during the early stages of construction and stockpiled at the screening plant about two miles from the dams.

The impervious material borrow pit was located on a hillside beside the main road about four miles from the dams. The material, a glacial till, was very dense in the natural state and there was no problem of excess moisture in the undisturbed materials in the pit even after heavy precipitation. The material was excavated in benches using power shovels.

Rockfill

Rockfill operations for the main body of the two dams were carried out by end dumping from the fills as they were advanced from the abutments. The fills were built from the north bank of the Desroches dam and, with the exception of the rock obtained from the coyote operation, from the south bank of the Bersimis dam.

Rock was placed in both dams in high lifts and a maximum height of 140 feet was reached in the Desroches dam. Segregation of the rock was apparent where the lift was high and, with the large rock concentrated at the bottom, special attention was required in the placing of the transitions, as mentioned later in this paper. The lifts were advanced with a minimum top width of 60 feet. This dimension was considered a minimum for efficient operation.

During the 1954 construction season the main body of rockfill on both dams was carried up to elevation 1240 with a top width of 60 feet. In the first stage of construction in the 1955 season the main rockfill was extended to its full width at elevation 1240. In the next stage the fills were raised to elevation 1280 where the final width of the main body is approximately 60 feet. In the final stages the rock was raised from elevation 1280 to the crest.

The surface preparation prior to placing succeeding lifts in the main rockfill body was done by bulldozers and rooters, and on some occasions by back hoes. Thorough sluicing of the surface was carried out after rooting. Considerable work was involved in preparing the top of a lift for the placing of a succeeding lift, and this confirmed one of the design considerations in the choice of the sloping-core type dam.

In contrast to the method of constructing the main body of rockfill, the upstream weighting zone was brought up in shallower lifts usually following the constructing of the impervious blanket and transition materials. Very little surface preparation was done on the weighting zone lifts.

Sluicing of the rock placed in the main body was carried out from cantilevered platforms located at each corner of the dumping face. Under a pressure of 125 psi the jets from the monitors were trained back toward the dumping face and met the rock as it first came in contact with the slope. This operation is shown on Fig. 13. The volume of water used was generally equivalent to four times the volume of rock placed.

Special precautions were exercised in dumping rock on the north bank of the Bersimis dam. Since the overburden was covered with transition materials, it was necessary to avoid the displacement of this material by falling rock. The rock was dumped against the abutment rather than down the bank at this location.

An attempt was made to place a well graded fill throughout by classifying the rock size in the trucks as they left the quarry. With the high lifts used in the lower parts of the dams however, it was difficult to prevent segregation of the rock and the practice met with limited success. On the basis of experience gained with the high lifts, it is believed that there is some advantage in specifying the placing of a finer rock in an upstream zone in order to limit the size of voids adjacent to the transition layers.

Transition

Transition materials were used both upstream and downstream of the core and also under the rockfill on the north bank of the Bersimis dam. These materials were spread and compacted by bulldozer. To prevent bulking of the

FIG. 13 BERSIMIS DAM – ROCKFILL

FIG. 14 PIPE FOR PLACING TRANSITION MATERIAL

fine transition material, excess water was added during the compacting operation.

Due to traffic problems across the impervious fill, an alternative method of placing the coarsest transition was adopted for the zone on the downstream side of the impervious core. The alternative method consisted of dumping the materials through 26-inch diameter pipes located on the upstream slope of the main rockfill. The momentum of these materials in the pipes was checked by heavy chains which hung at the outlet of the pipes above the placing level. This alternative was found to be reasonably satisfactory. A picture of the arrangement of a pipe on the rockfill slope is shown on Fig. 14.

As stated previously, segregation in high lifts in the rockfill caused concentration of large rock at the bottom of the slopes. This necessitated special placing of the coarsest transition material by hand in filling some of the larger voids.

Impervious Core

In general, the impervious core material was transported by truck from the borrow pit, dumped on the fill, and spread by bulldozers. During the period of greatest congestion in 1955, however, trucks were driven on to a ramp on the Bersimis dam and emptied into scrapers. These were then driven on to the fill and the material was dumped and spread in the same operation. In this way trucks hauling the earth fill did not interfere with the placing and compaction of impervious fill.

The impervious fill was spread in layers 8 inches thick. Boulders exceeding 5 inches in diameter were removed from the fill by hand after the spreading operation, and these were dumped in the coarse transition zone or in the upstream weighting zone. A large number of boulders were removed from the fill in this way.

Addition of a small amount of moisture was frequently required on the impervious fills, and it was added by spraying with hoses during the spreading operation. The water content was maintained below the standard Proctor optimum during the 1954 construction season when the lower part of the dam was constructed. During the final construction season however, as the fill rose to higher elevation, the water content was kept at or above the optimum.

Since construction of the impervious zone fell behind schedule in 1954, placing was continued as late as possible in the autumn season. As early as mid-October freezing weather interfered with operations during the night shift and salt was utilized on several occasions to make it possible to continue the placing operation. Calcium chloride was dissolved in the water which was sprayed on the fill to bring the water content to the specified value, and this usually made it possible to continue the placing operation with up to 12 degrees of frost.

Compaction of the impervious fill was carried out by means of pneumatic-tired vehicles, which usually were loaded tandem-wheeled, 10-ton hauling units. Compaction very near to standard Proctor density was maintained throughout.

Due to the grading of the impervious core material, it was not practicable to attempt to carry out density tests in accordance with some of the established procedures. Instead a method was adopted which involved the excavation and transportation to the field laboratory of a chunk of compacted fill. A specimen was broken from this mass, weighed, and then immersed in oil to determine its volume.

Fig. 15 shows an over-all picture of the Bersimis dam under construction.

Summary of Quantities and Rates of Construction

Table 4 contains a summary of the quantities of materials used in construction of the dams.

TABLE 4

Summary of Quantities in Cubic Yards

Material	Bersimis Dam	Desroches Dam	Total
Rockfill	2,575,000	1,081,000	3,656,000
Transition	713,000	222,000	935,000
Impervious	383,000	124,000	507,000
	3,671,000	1,427,000	5,098,000

The weekly rates of placing materials were as follows:

Rockfill	80,000 cu yd
Transition	30,000 cu yd
Impervious	15 - 20,000 cu yd

Performance

Reservoir Elevations

Filling of the Lac Cassé reservoir began in 1955 and by the end of construction in November, 1955 its elevation was 1170 feet. Continued storage of water steadily raised the elevation to its normal maximum level of 1305 feet by November, 1957. The level was then gradually drawn down to elevation 1288 feet by late April, 1958, and since the commencement of the spring flood it has now risen again to 1305 feet.

Settlement Records

Settlement monuments were established on the crests of both dams in early 1956 for the measurement of vertical and horizontal movements. Between May, 1956 and February, 1958 maximum settlements of 1.9 inches and 1.8 inches respectively have been recorded in the Bersimis and Desroches dams. In both dams the settlements have varied directly with the height of fill. Horizontal movements during this period have been negligible or within the limits of instrumental error.

Seepage

During and subsequent to the filling of the reservoir a close observation has been maintained of the seepage emerging at the downstream toes and

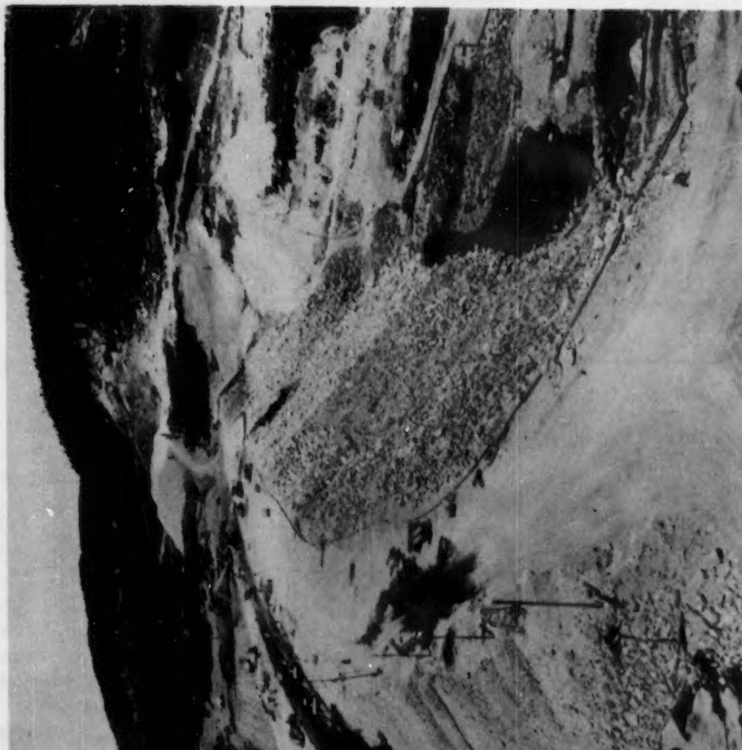


FIG. 15 BERSIMIS DAM - GENERAL VIEW SHOWING CONSTRUCTION

abutments. A very small amount of seepage has occurred through the south rock abutment of the Bersimis dam but the quantity is not considered to be injurious and no attempt has been made to stop it. The seepage through the Bersimis north bank has not materialized to a discernible degree, and on the basis of the experience gained thus far it is believed that no seepage problem will develop there. The existence of seepage through the dam proper is difficult to establish, but no indication of abnormal water percolation through either dam has yet appeared.

ACKNOWLEDGEMENT

The authors are grateful to the Quebec Hydro-Electric Commission for permission to present this paper and to its Engineers who have supplied much of the information concerning the Bersimis dams. The authors also wish to thank H. G. Acres & Company Limited and their colleagues for their assistance in the preparation of this paper.

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Paper 1741 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: THE DERBENDI KHAN DAM

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(Proc. Paper 1741)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Described in this paper are the problems pertaining to the design of a 450-foot high central core rockfill dam. Foundation excavation and preparations for diversion are now under way. The Derbendi Khan project consists of the rockfill storage dam, chute spillway, outlet works, and provisions for a future powerhouse. Diversion of the river was provided for by two tunnels that will serve the permanent outlet works and powerhouse after construction.

A thorough field exploration and materials testing program provided a sound basis for the selection and design of the dam. Major problems were anticipated and are being provided for as a result.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1741 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

1. Pres. Harza Eng. Co., Chicago, Ill.

General Description

The Derbendi Khan rockfill dam is located on the Diyala-Sirwan River approximately 230 kilometers northeast of Baghdad, Iraq, and ten kilometers downstream from the confluence of the Tanjero River. The project is owned and will be operated by the Development Board of the Government of Iraq. The location of the project and its drainage are shown in Figure 1. At the Derbendi Khan site the river flows through a three-kilometer long gorge in the Baranand Dagh mountain range. This range is part of the northwest-southeast chain of mountains forming the Persian arc. The site is located about one kilometer above the downstream end of the gorge.

The primary function of the Derbendi Khan dam will be the storage and regulation of the Diyala-Sirwan River for irrigation use. The greatest value and need for water in Iraq is for agricultural irrigation in the fertile but dry plains of the Tigris and Euphrates valley. Also the regulation of the river and the head created by the dam will make feasible the generation of hydroelectric power at Derbendi Khan as a secondary benefit of the project. Some measure of flood control will be available at Derbendi Khan, although not without slight reduction in irrigation storage benefits. As presently planned, the flood control is considered of secondary importance.

Above the dam site the Diyala-Sirwan River has a drainage area of about 17,850 square kilometers (6900 square miles). The 30-year average annual flow passing the dam site is 3.8 milliards of cubic meters (3,100,000 acre-feet). The gross storage of the Derbendi Khan dam is 3.0 milliards (2,430,000 acre-feet). A typical hydrograph is shown on Figure 2. The maximum flood of record is very close to 4000 cubic meters per second (141,500 cfs). During the months of January through May the river flows are generally in excess of 200 cubic meters per second (7100 cfs). The flows from June through December are usually less than 100 cubic meters per second dropping to less than 50 cubic meters per second in November and December.

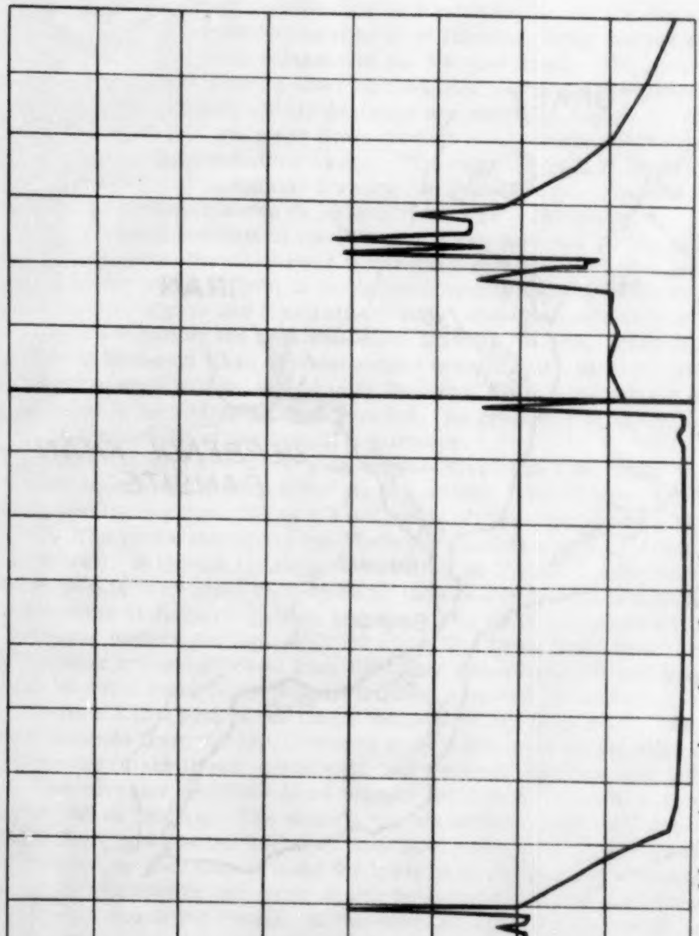
Figure 3 is a plan of the major structures of the project. The rockfill dam extends from the left abutment across the river to the right abutment, where the intake tower, guide wall, and the chute spillway are located.

Two circular concrete-lined tunnels provide for diversion around the right end of the dam. The tunnels are six meters (19.7 feet) and nine meters (29.5 feet) in diameter and after they have served for diversion they will be converted so they may be used for irrigation and power releases. This will be accomplished by extending shafts from the tunnels to the intake tower at upstream end of the tunnel. At the downstream end a transition will be placed and smaller diameter penstocks will branch off and connect with the outlet works and future powerhouse. Steel liner will be used in portions of the six-meter and nine-meter tunnel and the penstocks.

The hydraulic configuration of intake tower, guide wall, and chute spillway was established as the result of model tests carried out at the St. Anthony Falls Hydraulic Laboratory under the direction of Dr. Lorenz G. Straub. Model studies of the initial designs of the intake tower and guide wall indicated minor disturbances in the flow approaching the spillway. However, this was enough to generate a series of wave patterns and led to unstable flow conditions in the chute. Alterations of the guide wall, streamlining of intake tower, and leveling the approach channel floor solved this problem. The bucket at the downstream end of the chute is designed to turn the water away from the slope and into the stream bed downstream from the concrete structures. At

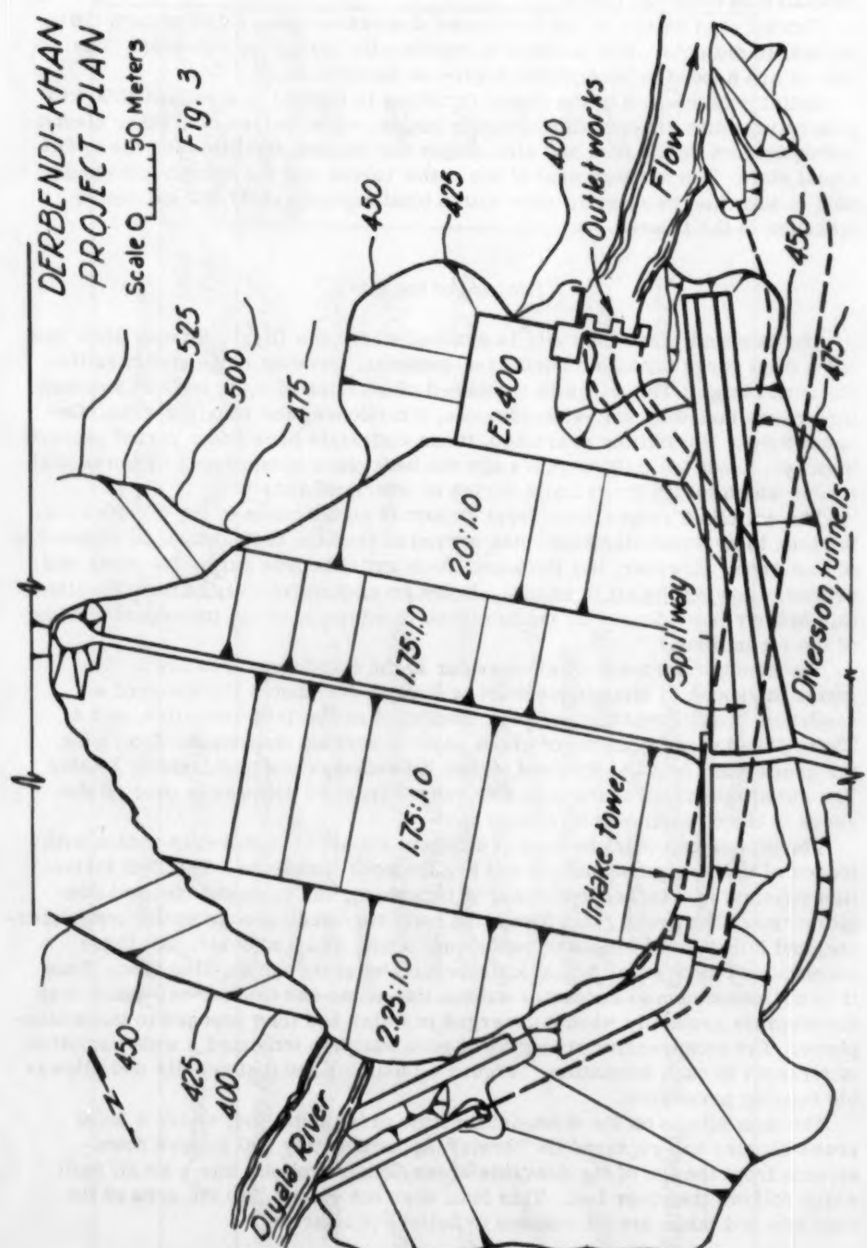

TURKEY
SYRIA
IRAN
IRAQ
Mosul
Greater Zab River
Lesser Zab River
Diyala River
DERBENDI KHAN DAMSITE
Euphrates
Baquba
Baghdad
Tigris River
River
LOCATION MAP
0
200
Kilometers

Fig. 1



TYPICAL HYDROGRAPH - YEARS 1941 & 1942
DIYALA RIVER - DISCHARGE SITE

Fig. 2


DERBENDI-KHAN
PROJECT PLAN
Scale 0 50 Meters
Fig. 3
525
500
475
450
425
400
El. 400
Outlet works
Flow
2.0:1.0
1.75:1.0
1.75:1.0
2.25:1.0
Spillway
Diversion tunnels
Intake tower
Diyala River
N
450
475
450
425
400

maximum discharges the jet will land in the stream bed about 150 meters downstream from the bucket.

Three outlet valves of the fixed-cone dispersion type, 2.438 meters (96 inches) in diameter, will be used to regulate the irrigation releases. The valves are hooded to control the degree of dispersion.

Initial construction of the power facilities is limited to a pair of 1000 kva generating units for operating the gate hoists, outlet valves, and other electrical equipment on the dam and also supply the housing facilities for the operational staff. The arrangement of the outlet valves and the service generating unit is such that two larger units with a total capacity of 37,000 kw can be installed in the future.

Geology of the Site

The Derbendi Khan dam site is situated where the Diyala-Sirwan River has cut a deep gorge through a northwest-southeast trending ridge known as the Baranand Dagh. This ridge is composed of a series of thick beds of sedimentary rocks including marls, sandstones, limestones, and conglomerate. Deposited over 100 million years ago, these materials have had a varied geologic history. About ten million years ago the beds were compressed by horizontal forces which folded them into a series of anticlinal ridges.

The anticlinal ridges have great economic significance in the Middle East, because their dome-like shape has served to trap the tremendous oil resources of that area. However, the Baranand Dagh anticline was cut by the river and eroded, allowing the oil to escape. In the immediate vicinity of the dam site, this history is evidenced by the presence of bitumen, which impregnates some of the formations.

The geologic formations which occur in the foundation area are a 335-meter thickness of bituminous marl, a 200- to 300-meter thickness of a sandstone-marl-limestone complex, identified as the Buff formation, and a 70- to 150-meter thickness of green marl of varying soundness. Overlying the green marl is a fine grained, dense limestone, usually massively bedded. The dip of the strata at the dam site ranges from 35 degrees to over 80 degrees in the downstream direction.

The bituminous marl formation consists mainly of calcareous shales with lenses of calcareous sandstone and argillaceous limestone. The Buff formation consists of interbedded lenses of limestone, shale, sandstone, and conglomerate. The green marl formation consists of calcareous shale, conglomerate, and interbedded lenses of sandstone, shale, and limestone. All three formations can be identified as calcareous and of the compaction type. Thus it is not uncommon to encounter substantial areas and thicknesses which may disintegrate gradually when submerged in water and then exposed to the atmosphere. The compressive strengths of core samples indicated a wide variation of strength in each formation. Figure 4 tabulates the test results and allowable bearing pressures.

The dam site is on the downstream limb of the anticline, where a local cross-flexure has ruptured the strata. Approximately 200 meters downstream from the toe of the dam this cross-flexure passes into a small fault which follows the river bed. This fault does not extend into the area of the dam site and there are no reasons to believe it is active.

ROCK FORMATIONS
Test Results and Allowable Bearing Pressures

Formation	Description	Unconfined Compression Tests, Min. and Max., T/SF		Allowable Brg. Pressure, T/SF
Upper Green Marls	Compacted, calcareous, clayey shale.	140	500	Not used as a foundation rock
Lower Green Marls	Shale, sandstone, and sandy limestone.	445	1030	25
Buff "D"	Limestone, sandy limestone, sand stone, and shale.	270	1070	25
Buff "C"	Nodular limestone with lenses of shale and sandstone.	190	1000	25
Buff "B"	Clayey and sandy shale, with beds of nodular limestone.	85	515	10
Buff "A"	Calcareous sandstone with conglomerate lenses.	660	1025	25
Bituminous Marls	Compacted calcareous and argillaceous shale, with beds of nodular limestone.	9	265	7.5

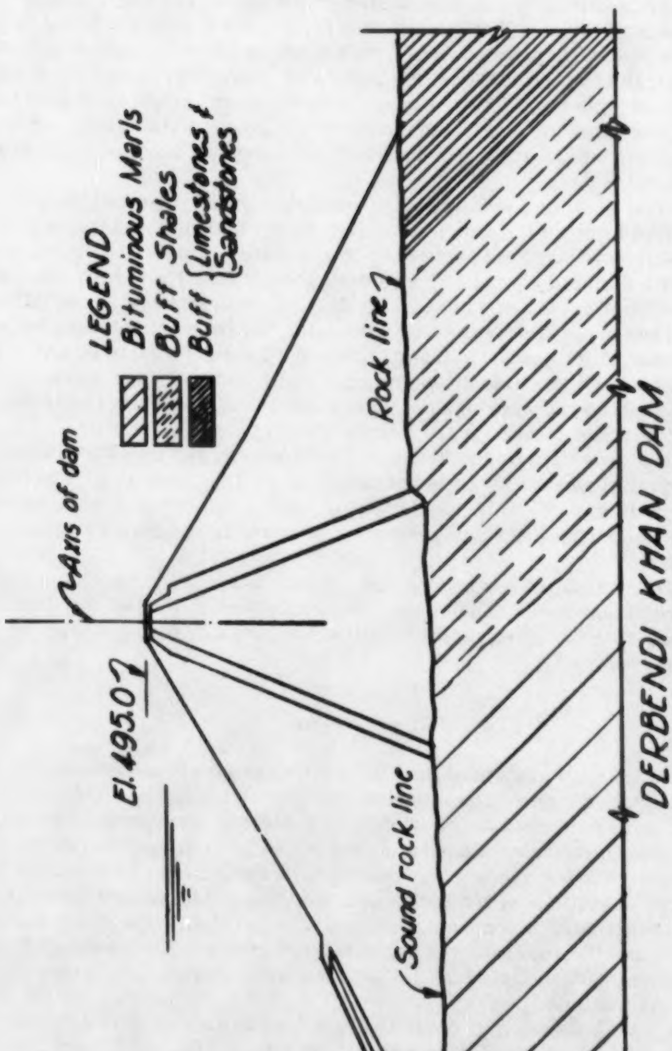
Fig. 4

The gorge topography of the Derbendi Khan site is emphasized by the nearly vertical cliffs of Qarah Chauq limestone, which form the top of the gorge walls on both abutments above the dam and spillway and can be noted in Figure 5. From the foot of the limestone cliffs talus slopes extend down to the river banks. The talus has in almost all areas weathered to a soil which supports sparse vegetation to the extent permitted by the semiarid climate.

An analysis of the geologic and topographic conditions was the basis of the final selection of the axis of the rockfill dam and spillway. Of the aforementioned succession of formations, the bituminous marl is under the upstream half of the rockfill dam and for a slight distance under the core in the river bed. The Buff formation in general underlies the core, the upstream shell at higher levels of the abutments, and the entire downstream shell of the dam, as indicated in Figure 6. The intake tower is also located within the Buff formation on a 20-meter thickness of sandstone. The guide wall, spillway ogee, and the upstream portion of the spillway chute are located on the marls and limestones of the Buff formation. The downstream portion of the spillway chute and the terminal bucket are located on the more competent portions of the green marl.

VIEW LOOKING DOWNSTREAM
DERBENDI KHAN PROJECT

Fig. 5



DERBENDI KHAN DAM
GEOLOGIC SECTION

Fig. 6

On the left abutment the overburden, formed by the weathered talus, ranges in depth from about two to 55 meters, the average depth being in the order of ten to fifteen meters. The overburden consists usually of debris from the Qarah Chauq limestone in a matrix of clay. The limestone debris ranges from fragments a few centimeters in diameter to blocks over five meters in diameter, generally separated by the clay matrix. In its natural state the matrix is dry and hard and the overburden, as a mass, is relatively impermeable.

On the right abutment the overburden varies considerably in character and thickness depending on the underlying rock formation. In the areas underlain by the bituminous marl the overburden is generally a clay soil derived from the marl with isolated blocks of limestone, usually less than one meter in diameter. The overburden varies in thickness from eight meters to 38 meters.

In the areas underlain by the Buff formation the limestone debris from the overshadowing cliffs is predominant, forming a limestone talus in which the interstices between the limestone particles are filled with clay and silt making the deposit relatively impermeable. The talus has a maximum thickness of approximately eight meters at the base of the slope.

In the area underlain by the green marl formation, the overburden is a green residual clayey soil with isolated blocks of limestone usually less than one meter in diameter. This soil is very plastic when wet and there is evidence of creep during the rainy season. Thickness is estimated to average eleven meters.

In the river channel the deposits consist primarily of blocks of limestone and other resistant rocks ranging up to seven meters in diameter. Explorations indicate that the river deposits will not be greater than few meters thick on the average.

Design of Dam

In the preliminary studies of the Derbendi Khan site considerations of the various alternative types of dam, which might be suitable for the foundation conditions, were narrowed to the studies of a rockfill and concrete gravity type dam. Comparative cost estimates tended to favor the rockfill type, but in an area where price precedence had not been established by previous construction, the magnitude of the estimated cost difference was not sufficient to warrant elimination of a concrete dam study. Accordingly the designs for the rockfill dam and the concrete gravity dam were carried along through the various preliminary stages including the preparation of contract drawings and specifications for both.

Tenders were published in April 1955 for both a rockfill dam and concrete gravity dam. The bids were received and opened on August 31, 1955. After thoroughly analyzing, interpreting, comparing, and evaluating the complex pattern of bids from six contracting combines, it was found that the rockfill structure was nearly fifteen percent cheaper than the lowest bid for the concrete gravity dam, representing a savings of approximately eight million dollars. This substantial differential in favor of the rockfill dam eliminated any further consideration of the concrete gravity dam.

During the preliminary studies of the rockfill shell with impervious core type of structure, conferences and consultations led to the following conclusions:

1. The unquestionable safety of the dam is of utmost importance, hence the design should reflect a reasonable degree of conservatism. The principal factors dictating such caution were the height of the dam, the heterogeneous nature of the foundation materials, and the somewhat limited area of strong rock, upon which to construct the spillway.

2. After a consideration of sloping core and central impervious core types of rockfill dams, the central core type was selected. The reasons for this selection were based on:

a. The low friction strength of the available core material would necessitate flatter slopes for the upstream shell of a sloping core type dam than for a central core type dam. Thus for a given upstream slope the central core type had a greater upstream stability. Flattening the upstream slope would increase the cost of the intake structure and guide wall.

b. In order that the major portion of the core be in contact with the Buff formation, only a central core could be adopted.

c. The central core contact area with abutment is greater than a sloping core. Also the full weight of the dam is applied to the contact area providing maximum safety against seepage. The pressure exerted along the foundation line of a sloping core is considerably less.

The above factors were very desirable in a more than 400-foot high rockfill dam.

3. Core side slopes of about 0.3 horizontal to 1.0 vertical were selected in preference to thicker sections. A thicker core would require additional shell material to provide an equivalent factor of safety against sliding under the condition of sudden drawdown.

4. The base of the impervious core section and at least the first or finest of the filter zones would be carried to sound and impervious rock throughout the entire foundation.

5. Pending exposure of the excavated foundation, the use of a small narrow cutoff trench excavated along the line of the proposed grout curtain was considered unnecessary; likewise, the pouring of a concrete cap to provide some support against uplift grouting pressures during first stage grouting was deemed unnecessary.

6. It was agreed that the downstream stability section of the dam would consist essentially of dumped quarry-run rock with minimum average downstream slopes of 1.75 to 1 for the upper two thirds of the slope and 2.0 to 1.0 for the lower one third of the downstream slope. It was understood that the slopes would be reviewed after stability computations of the relative safety against sliding along the foundation were completed.

7. The slope or slopes of the upstream section would be determined by stability calculations, such calculations to take into account resistance to sliding along the contact of the rockfill shell and the shale foundation as well as sliding within the embankment shell. These computations were based on use of quarried rock or stream bed snads and gravels for the shell material.

8. The crest section as wide as dictated by construction requirements.

Starting with the previously described criteria the resulting rockfill section was analyzed for the following conditions:

Case I. Construction condition: embankment completed to maximum elevation 495 meters and no water in the reservoir.

Case II. Construction condition: top of embankment at elevation 423 meters as required for diversion during the wet season, and with no water in

reservoir.

Case III. Normal operating condition: reservoir water level at top of gates, elevation 485 meters.

Case IV. Rapid drawdown: normal operating level elevation 485 meters drawdown to maximum drawdown level elevation 434 meters.

Case V. Normal operating condition: reservoir at elevation 485 meters with the added forces due to a horizontal earthquake acceleration of 0.10 gravity.

Case VI. Catastrophic flood condition: all gates raised and the reservoir level within 1.5 meters of the top of the embankment, pool at elevation 493.5 meters.

The maximum section at the river and other critical sections at the abutments were analyzed for stability. The analyses were made by the "sliding wedge" and "slip-circle" methods. Of these two methods the "sliding wedge" analysis is considered most applicable and will present more accurately the stability of a central core type rockfill dam. A "slip-circle" analysis will give a higher factor of safety under similar loading conditions. This can be explained by noting that the slip-circle only passes through a small portion of the core material and thus does not reflect the effect of the high pore pressures in the core on the stability of the shell material. A comparison of Figures 7 and 8 will show that the "sliding-wedge" analysis takes into account the full effect of the driving force created by the pore pressures in the core.

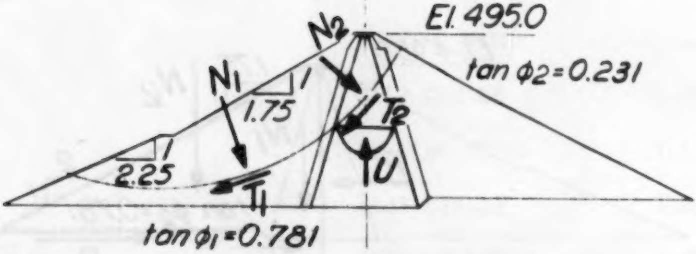
The most critical situation was with loading Case V with reservoir at elevation 485 and earthquake acceleration equal to 0.10 gravity, in which instance the downstream slope when analyzed by the sliding-wedge method showed a minimum factor of safety of 1.22. Factors of safety for the upstream and downstream slope analyzed for various loading cases are tabulated in Figure 9. Considering the extreme infrequency of the loading defined in Cases V and VI, the safety factor values of 1.22 and 1.30, respectively, were considered acceptable. A factor of safety of 1.47 for the downstream slope under normal operating conditions (loading Case III), may be considered as an index of the reserve stability for normal operating conditions.

After minor modifications to foundation conditions, diversion requirements, and the limitation set by the appurtenant structures, the maximum section at the river bed was established as shown in Figure 10.

The upstream shell and the impervious core excavation will be carried down to sound rock while the downstream shell will be permitted to rest on slightly weathered rock. The exposed slopes of overburden excavation will be 1.35 horizontal to 1.0 vertical. To minimize the excavation volume a five-meter thickness of overburden was permitted to remain on the left abutment near the top of the dam. As the dam is built in this area the overburden will be removed permitting placement of the fill on sound rock. The excavation will be carried out in benches as shown on Figure 11.

Because some of the marl areas were susceptible to drying and checking when exposed to the atmosphere for long periods of time it was specified that all marl foundations be coated with a bituminous sealer immediately after excavated to grade. Area under the impervious rolled earth core is to be treated with a slush grout in lieu of the bituminous sealer. In cases where the slope of the foundation is too steep for application of slush grout the exposed rock will be gunited to a minimum thickness of fifteen millimeters.

Condition IV - Pool drawdown from El. 485.0 to El. 434.0



Driving forces:

$$\Sigma T = 11610^K$$

Resisting forces:

$$R_1 = \Sigma N_1 \times \tan\phi_1 \qquad R_1 = 14770^K$$

$$R_2 = \Sigma(N_2 - U)\tan\phi_2 \qquad R_2 = 678^K$$

$$cL = \underline{972^K}$$

$$16420^K$$

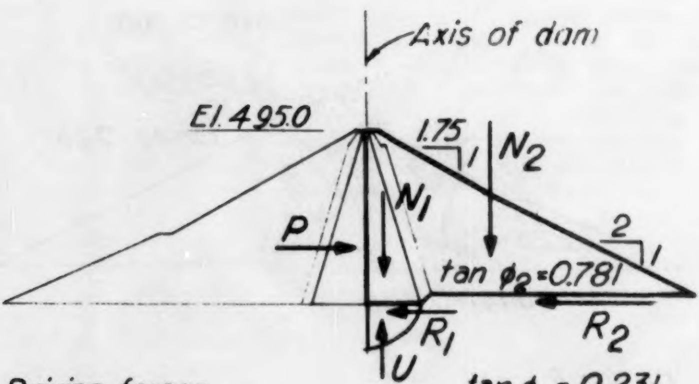
Factor of safety:

$$F.S. = \frac{16420}{11610} = 1.41$$

Fig. 7

Condition III - Max. operating pool
El. 485.0

MAX. SECTION



Driving forces: $\tan \phi_1 = 0.231$

$P = 8030^K$
$U = 2215^K$

Resisting forces:

$R_1 = (N_1 - U)\tan\phi_1$ $R_1 = 1345^K$

$R_2 = N_2 \times \tan\phi_2$ $R_2 = \underline{10060^K}$

11405^K

Factor of safety:

$$F.S. = \frac{11405}{8030} = 1.42$$

Min. F.S. = 1.22 (Earth quake) Fig. 8

DERBENDI KHAN PROJECT

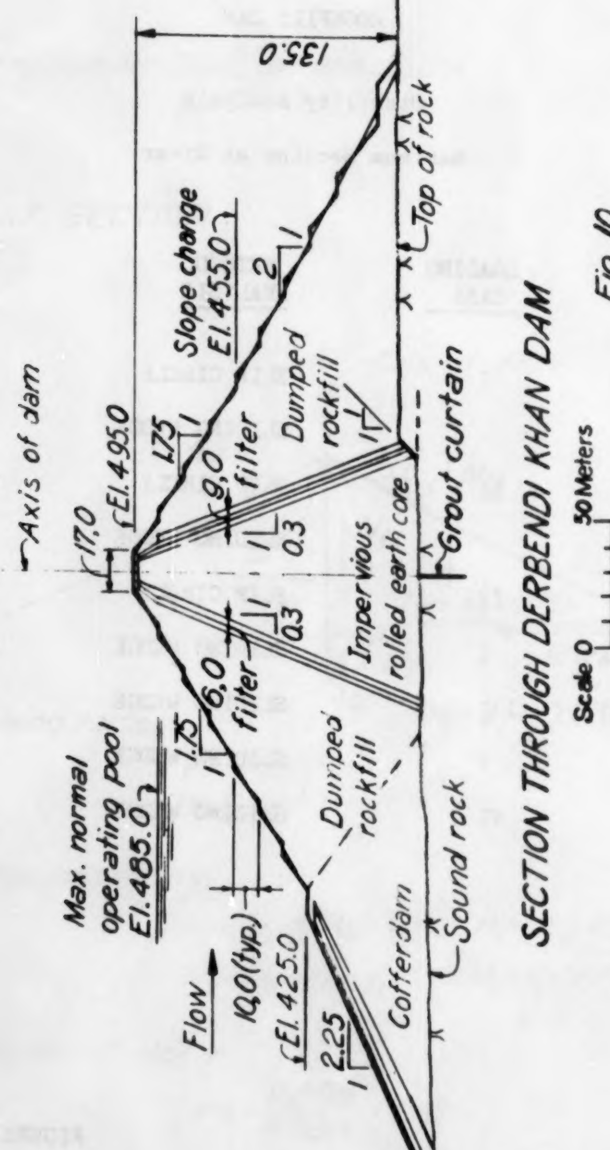
ROCKFILL DAM

Stability Analysis

Maximum Section at River

SLOPE	LOADING CASE	METHOD ANALYSIS	SAFETY FACTOR
UPSTREAM	I	SLIP CIRCLE	2.21
		SLIDING WEDGE	1.41
	II	SLIP CIRCLE	1.82
		SLIDING WEDGE	1.47
	IV	SLIP CIRCLE	1.41
DOWNSTREAM	I	SLIDING WEDGE	1.83
	III	SLIDING WEDGE	1.47
	V	SLIDING WEDGE	1.22
	VI	SLIDING WEDGE	1.30

FIGURE 9



SECTION THROUGH DERBENDI KHAN DAM

Fig. 10

LEFT ABUTMENT EXCAVATION
DERBENDI KHAN PROJECT

Fig. 11

To facilitate construction of the dumped rockfill shells, the exterior slopes may be formed in ten-meter benches as shown in Figure 10. The width of these benches is designed so that the average slope, i.e. from the center to center of each bench, will be equal to the design slope. The slope between each bench will be the natural dump slope of 1.35 horizontal to 1.0 vertical.

To provide maximum compaction of the dumped rockfill shell the rock will be end-dumped and sluiced with hydraulic monitors. The monitors will be directed so that fines are forced down into the voids between the larger rocks. This will reduce the voids and keep future settlements to a minimum. The ratio of sluicing water volume to rockfill is specified as three to one. The lifts for end dumping are specified as 30 meters (100 feet) maximum and ten meters minimum.

The transition from the fine core material to the coarse rockfill is specified in two stages on the upstream side and three stages on the downstream side. Each filter stage is three meters wide to permit placement by end dumping from trucks and spreading in layers by mechanical methods. The filters are to be compacted along with core material and in the same manner.

The core material is to be placed in 30-centimeter or 15-centimeter layers and compacted with either rubber tired or sheeps foot rollers, respectively. The moisture content is to be controlled to develop the maximum practicable density with four passes of the compaction equipment.

The grout curtain is shown on Figure 12. The deep cutoff grout curtain extends from the left abutment of the dam to the spillway structure on the right bank where it connects with the grout curtain under the gallery in the crest structure of the spillway. The grout curtain terminates beyond the right end of the spillway at elevation 495.0 meters (the crest of the dam). The stage grouting method by zones will be used to develop the grout curtain. Each hole is to be washed and pressure tested prior to the introduction of grout. The following table shows the specified grouting criteria for each stage.

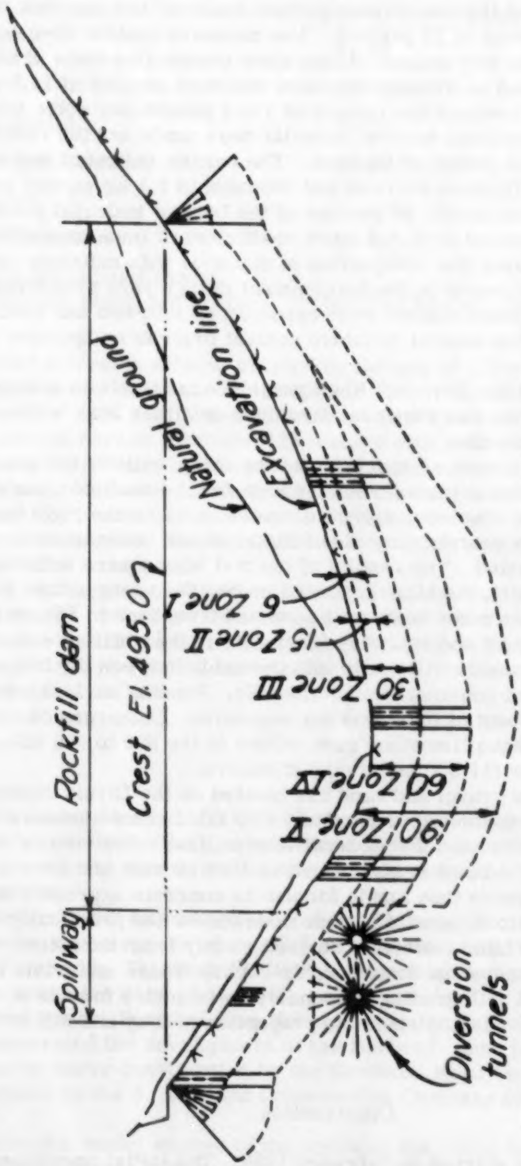
Stage	Depth in Meters	Maximum Pressure	Design Spacing Meters
I	6	1.0 kg/cm^2	1.60±
II	15	3.5 kg/cm^2	1.60±
III	30	7.0 kg/cm^2	1.60±
IV	60	7-14 kg/cm^2	6.50±
V	90	14-21 kg/cm^2	—

As each stage of grouting is completed the pressures for the succeeding stage may be increased. Stage I serves primarily as a low pressure, foundation consolidation operation.

In the foregoing tabulation Stages II and III are further extensions for the grout holes started as Stage I. Stage IV constitutes an exploration and a check by extending approximately every fourth hole of the Stage III pattern. Stage V comprises random spaced check holes, primarily for further exploring leaky areas detected by stage operations.

Constructional Materials

Exploration for suitable construction materials revealed that soil suitable for use as an impervious core, quarriable rock for constructing the shells of the dam and a supply of sand and gravel which could be processed for use as filter material were available within economic haul distances. The soil borrow area is located on a plateau approximately 1-1/4 kilometers from the dam site at an elevation of about 400 meters above sea level. A series of test pits defined the soil profile as having an average of ten centimeters of topsoil below which there is a zone of loamy sand averaging 0.6 to 0.8 meters in thickness, which in turn is underlain by a third zone similar to the loamy sand, except for the presence of calcium carbonate nodules formed by reprecipitation of the calcareous materials leeched from the overlying zones. Our present plans are to use the second zone with an average thickness of 0.7 meters. It may well be that after the borrow area is opened some area of the third zone may not have an excessive percentage of calcareous nodules. If such is the case, these materials may be used for the impervious core.



SECTION ALONG GROUT CURTAIN
DERBENDI KHAN DAM
Fig. 12

Laboratory tests of the impervious borrow material indicate that it has a natural moisture content of 22 percent. The moisture content does not appear to decrease during the dry season. Laboratory compaction tests of the borrow material disclosed an average optimum moisture content of 14.5 percent and a corresponding average dry density of 116.5 pounds per cubic foot. Triaxial tests of the compacted borrow material were made and the results incorporated in the final design of the dam. The results indicated that an average friction value of thirteen degrees and cohesion of 1.1 kg/square centimeter could be used. Approximately 80 percent of the borrow material passed a 200 sieve and when compacted provided a low coefficient of impermeability. Further testing revealed that compaction of material with moisture contents above optimum would result in the development of high pore pressures. Therefore, field moisture control must carefully be followed and methods developed to reduce the natural moisture content prior to compaction in the impervious core.

The steep walls of the Derbendi Khan gorge are favorable to quarrying operations and quarries can easily be developed on either bank within a kilometer or so of the dam site.

The Qarah Chauq limestone which forms the steep walls of the gorge is a moderately hard, dense to porous rock suitable for the shell of a rockfill dam or, when crushed, for concrete aggregate. A test area on the right bank was blasted to explore the quarriability of the material and the manner in which it would break when blasted. The results of the test blasts were extremely satisfactory. Generally, the blast produced no boulders larger than 0.2 cubic meters and retrievable sizes were well assorted from four to 75 centimeters. There were some slabby and splintery fragments in the smaller sizes, but inasmuch as these fragments will settle into the voids between the larger blocks the percentage was not considered objectionable. Bonding surfaces were clean and rough and the amount of fines was not excessive. Compression tests on cores of the Qarah Chauq limestone gave values in the 800 to 900 kilogram per square centimeter (11,500 psi) range at failure.

Suitable sources of gravel and sand are located on the Diyala-Sirwan River with the major deposit approximately five kilometers upstream from the dam site. It is estimated that at least four million cubic meters are readily available in a deposit several hundred meters wide and over a kilometer long. This is more than ample for use as concrete aggregate and filter material. The deposits consist of almost unstratified and practically unconsolidated alluvial and talus materials derived mainly from limestone with some chert, basic igneous and metamorphic rocks. These materials contain some 40 percent sand with grading reasonably satisfactory for use in concrete,but will require washing since the proportion of clay and silt is fairly high.

Construction

Major construction started in February 1956. The initial operations were completion of the access roads, construction plant preparations, excavation and completion of the diversion facilities.

Some of the basic unit bid prices for construction at the time of award are listed below:

Item	Unit Price/cubic meter
Rockfill	$1.75
Filters	$4.80
Core Material	$1.75
Open Common	$0.65
Excavation	
Open Rock	$2.30
Concrete (Including Cement, Forms, and Reinforcement)	$30.00 (Average All Classes)

The initial diversion scheme planned on the use of a low cofferdam (incorporated in the upstream shell with an impervious blanket) for dry season closure. This plan provided sufficient time to construct the upstream shell and central core to elevation 425 meters with sufficient downstream shell for stability. Thus, in this scheme the main core would serve as the wet season cofferdam and enable flows of 1600 cubic meters per second to pass through the two diversion tunnels. Including the effect of storage upstream of the cofferdam, floods of 2600 cubic meters per second could be handled without overtopping.

However, construction difficulties delayed the start of diversion and the initial wet season cofferdam scheme was modified. The present cofferdam scheme is shown on Figure 10. The effect of the modification was to reduce the amount of material to be placed prior to wet season diversion and thus minimize the delays.

SUMMARY

As of the end of March 1958 approximately 30 percent of the total excavation was accomplished. The diversion facilities are being readied for turning the river into the tunnels in the near future, when flood flows are no longer expected.

A tabulation of the salient features of the Derbendi Khan project is shown on Figure 13. A photograph of a model showing the completed project is shown on Figure 14. The model indicates the anticipated extent of the excavation above the dam.

The Development Board of Iraq through their First Technical Section have administrated the development of the Derbendi Khan project.

The dam is under construction by the Derbendi Khan Contractors, a joint venture headed by the J. A. Jones Construction Company of Charlotte, North Carolina.

The hydraulic model studies of the spillway and outlet works were performed by Dr. Lorenz G. Straub, M. ASCE, at the St. Anthony Falls Laboratory of the University of Minnesota.

Mr. I. C. Steele served as Consultant on the foundation and rockfill dam design.

Mr. D. W. Griffiths, Affiliate Member ASCE, is serving as Resident Engineer at the project.

Mr. D. P. Roberts, M. ASCE, served as the Project Manager in early stages of design. Mr. E. J. Beck, A.M. ASCE, is presently serving as Project Manager.

SIGNIFICANT DATA

DERBENDI KHAN ROCKFILL DAM

Drainage Area	*17850 sq. km.*	*6883 sq. miles*
Length of Reservoir	*30 km.*	*18.6 miles*
Max. Height of Dam	*135 m.*	*443 feet*
Width of Base	*513 m.*	*1684 feet*
Crest Length	*445 m.*	*1460 feet*
Total Volume of Fill	*7,000,000 cu. m.*	*9,156,000 cy.*

Material Characteristics

Impervious Core $\theta = 13°$ *Cohesion = 1.1 kg/sq.cm.*

Dumped Rockfill $\theta = 38°$

Fig. 13

MODEL OF COMPLETED
DERBENDI KHAN PROJECT

Fig. 14

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: NANTAHALA SLOPING CORE DAM[a]

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(Proc. Paper 1742)

ABSTRACT

The design of a 250-foot high sloping core rockfill dam, part of a hydroelectric project, was determined to facilitate construction with materials readily available at the site. The paper describes the site and discusses construction procedures and performance.

INTRODUCTION

The Nantahala Hydroelectric Development is owned and operated by the Nantahala Power and Light Company, a subsidiary of the Aluminum Company of America. It is situated on the Nantahala River, in southwestern North Carolina, where steep slopes permit a high head development in a relatively short reach of river. The development consists of an earth-faced rock-fill dam, 250 feet high, creating a reservoir with 126,000 acre feet of useful storage, a spillway, a diversion tunnel, a conduit from the dam to the powerhouse, consisting of tunnels and a penstock, and a single 60,000 hp. hydroelectric generating unit operating under a maximum static head of 1,008 feet. The construction of the project was started in July 1940 and completed in July 1942. It has been in successful operation since that date.

The dam site is at the upper end of the Nantahala gorge, where the river flows between a rocky promontory on the left bank and a 40-degree solid rock slope on the right bank. (Figures 1 and 2.) The rock at the dam site is a metamorphic sandstone, called arkose. While the bed rock has been folded it occurs at the dam site in massive strata with few cracks or seams. It is exceptionally water-tight. It is an excellent site for any type of dam, except an

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1742 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at the October 1948 ASCE Convention in New York, N.Y.

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Figure 1

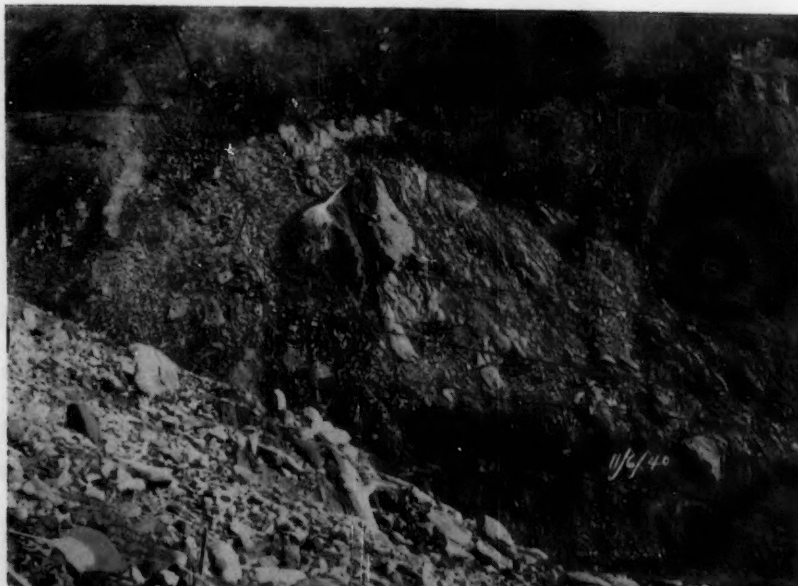


Figure 2

earth dam for which suitable material is not available. The dam site was acquired by the parent company in 1911. Since that date many different types of dams, including concrete gravity, combination gravity and arch, pure arch, multiple arch, Ambersen, and rock-fill have been designed for this site. Every type studied could be made to fit the site reasonably well, with varying degrees of effectiveness and economy. The rock-fill dam appeared particularly advantageous. Rock-fill dams as of 1935 had some type of a more or less rigid impervious upstream face which appeared likely to crack due to continued settlement of the rock fill. A flexible impervious face capable of adjusting itself to settlement appeared desirable.

Many of the lakes in northern United States and Canada have been formed by glaciers. The glacial moraine filling up old stream beds made effective dams. Generally speaking, they consist of coarse gravel, with progressively finer material upstream and are sealed at the upper surface by glacial silt. Most of them are surprisingly water-tight and have functioned since the glaciers disappeared, - a period of several thousand years. It appeared feasible to use the same technique in a man-made structure. Further study of the problem indicated that a suitable filter would prevent the migration of an impervious earth core through the pervious rock-fill and that a reverse filter covered with rock would protect the earth core from wave action and sudden drawdown.

Design

This conclusion was checked by a small model, consisting of a 12-inch steel pipe closed at the bottom with a perforated plate and at the top with a cap having a pipe connection. A layer of 2-inch crushed rock, 4 inches thick, placed on a perforated plate, represented the main body of the dam. Above this was placed a 10-inch layer of 1/2 inch to one inch gravel and a 14-inch layer of fine sand. Above the sand a 30-inch layer of compacted earth represented the impervious core. This impervious core was placed in layers and compacted by rodding equivalent to the compaction, which could be obtained by sheep's-foot rollers. Above the impervious section a reverse filter was placed, consisting of 4 inches of fine sand, 4 inches of 1/2 to one inch gravel, and 8 inches of crushed rock. Water under a head of 275 feet was admitted to the space at the top of the model and the performance of the impervious core and the filter noted. This model was kept under pressure for several months. The seepage, when it could be measured, checked the permeability of the core material as determined in the Soil Mechanics Laboratory. (When the air was dry, seepage evaporated and could not be measured.) No passage of the impervious material through the filter could be detected. When the model was dismantled after more than a year of operation, the impervious core was found to have been compacted into a dense hard mass, which contained little or no water. Other models were made and tested, with substantially the same result.

An earth deposit near the site, available for use as an impervious core, was tested and found satisfactory. All of the studies and tests indicated that an earth-faced rock-fill dam, built of materials readily available at the Nantahala site, would be a safe and economical structure.

The design of the earth-faced rock-fill dam for the Nantahala site proved to be relatively simple. (Figure 3.)

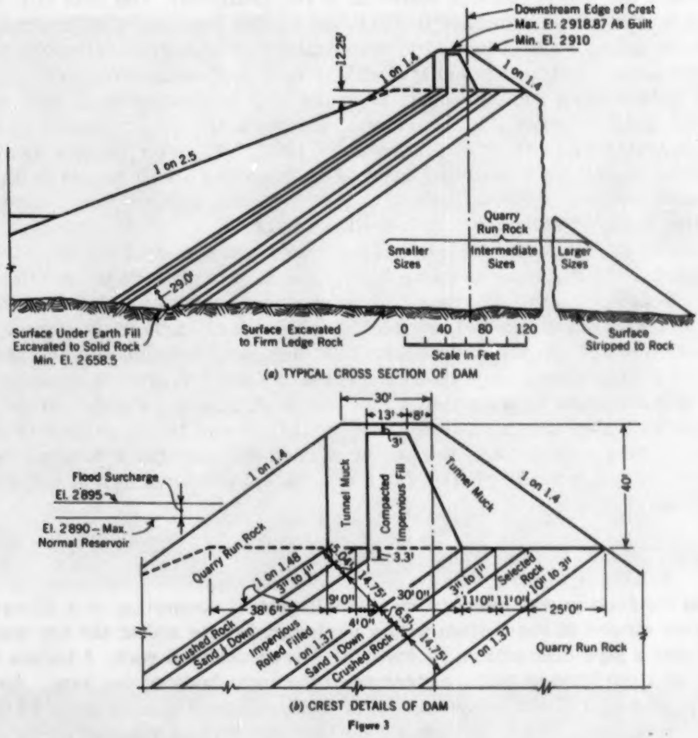


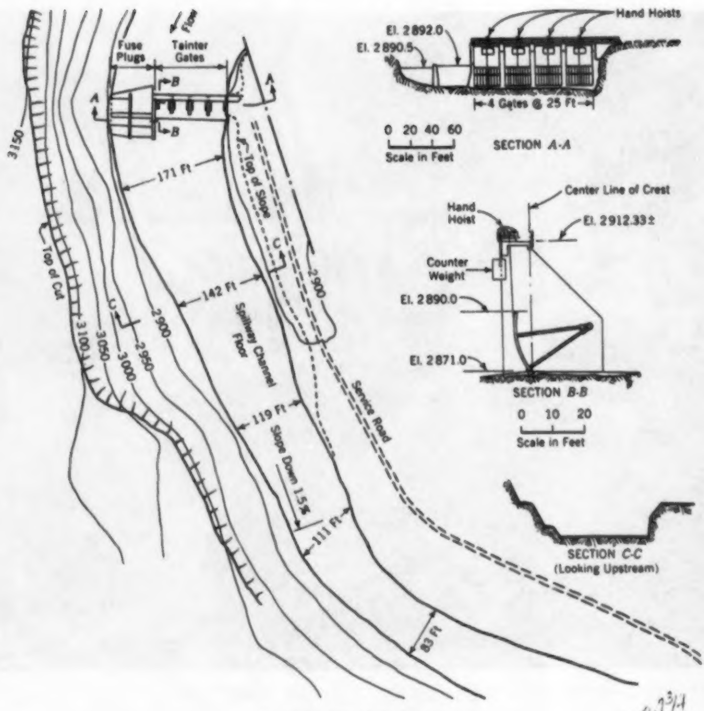
Figure 3

Hydrology

The drainage area above the dam site is 91 square miles. It is mountainous, with steep forest-covered slopes. The average run-off is 331 c.f.s. It is believed that large, sharp crested floods, of short duration are possible, although there is no evidence of a flood exceeding 15,000 c.f.s. since the valley was settled 150 years ago. The full reservoir, with water surface at Elevation 2890 (N. P. & L. Co. Datum) has a surface area of 1,631 acres.

A diversion tunnel, 16 feet in diameter and 1,400 feet long, through the left abutment, together with an earth and rock-fill cofferdam, 50 feet high, diverted the river during construction. The diversion works were designed for a flood of 7,500 c.f.s., or about one half the record flood.

An adequate spillway is an essential feature of all dams. The Nantahala spillway (Figure 4) was designed as a channel, cut through the solid, rock of the right abutment, having a discharge capacity of 57,500 c.f.s., equivalent to a peak inflow of 88,000 c.f.s, utilizing a surcharge of 13 feet. The spillway will take care of a flood peak of 978 c.f.s. per square mile, without



PLAN AND DETAILS OF SPILLWAY

Figure 4

L-2073/4 Pg. 5

Figure 4

encroaching on the 7 feet of freeboard above the surcharge level. (Figure 5.) The excavation from the spillway provided the rockfill for the dam. The spillway channel ends on a steep hillside, 400 feet downstream from the toe of the dam. The spillway discharge cascades into the original river channel.

The upper end of the spillway is closed by four hand operated Tainter gates, each 19 feet deep by 25 feet wide, and two sections of fuse plug between the end of the gate structure and the rock cut at the right of the spillway. The fuse plug adjacent to the Tainter gate structure is 33 feet 4 inches long, with its sill at Elevation 2871 and its crest at Elevation 2892, two feet above the crest of the Tainter gates. The fuse plug is composed of crushed rock, made watertight by a sand filter and earth blanket on the upstream slope. The earth blanket is protected against wave action by a filter and small rock cover. It is, in effect, a miniature earth-faced rock-fill dam, which will wash out when overtopped by more than one foot of water. The fuse plug adjacent to the rock cut is 25 feet long with its crest at Elevation 2890. 5. It is not intended to wash out when overtopped, but was provided primarily as a cushion to stop any rock which might spall off from the rock

Figure 5

face above it, without injury to the Tainter gates or fuse plug proper. The Tainter gates are counterbalanced and geared so that they can be opened or closed easily by one man. Hydraulic models of the spillway were made and tested. The final design was based on these tests.

The dam was designed as a triangular section rock-fill across the gorge, to be placed in one or more lifts as convenient for construction. (Figure 6.) Selective dumping placed the largest rock in the downstream part of the fill and the smallest quarry-run rock in the upstream part of the fill so that the voids in the main rock-fill would become progressively larger from the upstream face to the downstream face. Both the upstream and downstream slopes of the rock-fill were to be the natural angle of repose of the dumped rock. The first filter layer, 14.75 feet thick, was composed of selected rock not larger than 10 inches and not smaller than 3 inches. The second filter layer, 6.5 feet thick, was composed of crushed rock not larger than 3 inches and not smaller than 1/2 inch. The third filter layer, 6.5 feet thick, was composed of crushed rock fines smaller than 1/2 inch. The sizes specified and the grading of the filter materials produced by crushing fulfills the requirements that voids between the larger particles are filled by progressively smaller particles so as to prevent migration of the finer material. Theoretically, only a very thin filter layer would have been required. Actually, each

Figure 6

filter was designed thick enough to be placed without difficulty and to adjust itself to settlement of the rock-fill without danger of rupture.

Only a small amount of material suitable for an impervious rolled fill was available near the site. This limited the rolled fill to a thickness required to prevent excessive seepage. Laboratory tests determined that an impervious rolled fill, 29 feet thick at the bottom, where the head is 230 feet, and 13 feet thick at the top, would have a seepage rate of not more than 2.75 cubic feet per minute, which was satisfactory. The rolled fill makes contact with the foundation in a cut-off trench excavated to solid rock and grouted. A 5 foot layer of crushed rock fines minus 1/2 inch and a second 5 foot layer of crushed rock 3 inches to 1/2 inch provides a reverse filter, above which was placed a layer of quarry-run rock to protect the rolled fill from wave action and the effect of sudden drawdown. The action of the rolled fill when subjected to the stress due to sudden drawdown and the ability of the upstream rock-fill to resist this stress was uncertain. To avoid these uncertainties, quarry-run rock was added until the core was safe against sliding, when analyzed by the circular arc method generally used to determine the stability of earth-fills made of homogeneous material. The resulting upstream slope is 1 on 2-1/2. Since the material above the rolled fill is free draining, the design is amply safe. The space between the diversion

cofferdam and the main dam was filled with quarry waste, making the cofferdam an integral part of the main structure and increasing stability against sliding where drawdown stresses are the maximum. The upstream faces of the rock-fill, the filters, and the impervious rolled fill, are convex upstream so that as the dam settles the filters and the rolled fill will be further compacted. The top of the dam in the deepest section was cambered approximately 9 feet higher than the abutments to compensate for settlement.

Construction

After signing the contract in July 1940, the contractor began to assemble his forces and equipment and start work on clearing the dam site. To satisfactorily schedule the dam construction it was necessary to complete the river diversion as early as possible and most of the early work was devoted to the diversion tunnel. The contractor's forces began the portal excavation in September 1940 and had holed through in three and one half months. This was followed by construction of the concrete intake portal and placement of the concrete tunnel lining. Diversion was accomplished in March 1941.

As work progressed on the diversion structure, other forces completed the clearing and stripping of the dam site. Stripping was largely accomplished by ground sluicing using a total pump capacity of 8,400 gallons per minute at 275 feet head through light weight monitors. The larger materials which were not carried away by the river were excavated at the river's edge by power shovel and removed by truck. The entire foundation area occupied by the main rock-fill was stripped to rock. In the area covered by the impervious earth core, a cut-off trench was excavated to solid rock. Although it was necessary to excavate this trench 10 to 15 feet deep in one area, for the most part the cut-off trench excavation was limited to 2 to 6 feet. Upstream of the earth core, the foundation was only stripped of top soil and vegetation. Actually, however, there was little overburden remaining on the rock in this zone. In the cut-off trench area a grout seal was placed by drilling a series of percussion holes 25 feet deep and spaced on about 20-foot centers. These were followed by intermediate grout holes and further intermediate grout holes were required. In general, these holes were sealed at pressures ranging from 75 to 150 pounds per square inch. Following completion of this surface grouting, diamond core holes were drilled to a depth of 125 feet with holes spaced 100 feet apart in a single line near the center line of the cut-off trench. Many of these holes were drilled in an inclined position to intercept as many rock seams as possible. This curtain grouting was placed at 250 pounds per square inch pressure. Following completion at this spacing, intermediates were grouted until the holes were spaced 25 feet apart, and in some cases as close as 12-1/2 feet. Approximately 8,000 barrels of cement grout was required to make the foundation water-tight.

As soon as the diversion tunnel was ready for the river flow, a cofferdam was placed across the river channel and diversion accomplished. The cofferdam consisted principally of a rock-fill with an impervious blanket on the upstream slope, which later became part of the upstream toe of the main dam. With the river diverted, the balance of the stripping, grouting and foundation excavation was accomplished.

The spillway excavation was developed as a quarry and main source of rock for the dam. After stripping most of the earth and soft rock overburden,

the hill was benched in at the dam crest elevation using wagon drills. When the bench was wide enough to form a shelf to hold the muck produced by coyote blasting, the shooting was done from coyote holes. After completing all excavation above this bench, work was started on the channel using wagon drills and excavating in benches approximately 20 feet deep until the channel grade was reached. The spillway layout was adjusted from time to time so that a balance could be established between quarry cut and embankment fill. The rock from the spillway was used in the main rockfill, end dumping from a series of high lifts averaging from 50 to 130 feet in height. In January 1941 the main downstream rock-fill was started beginning on the right abutment at a level 130 feet above the river bed. When the river was finally diverted it was necessary to get the fill over the left abutment as quickly as possible to prevent a delay in placement of the rolled earth-fill. This was accomplished by ramping down and reducing the height of the lift to 100 feet. Following this, the entire area was brought up to the 130-foot level and thereafter carried up in high uniform lifts across the entire width of the dam. The original dump slope of the rock was estimated to be one vertical to 1.4 horizontal and was thus shown on the contract drawings. However, actual observations during placement indicated that the rock was dumping to slopes of one vertical to between 1.30 and 1.35 horizontal. It was therefore necessary to regulate the placement of the rock lifts so that the actual placed downstream slope would approximate the slope shown on the drawings.

All quarry rock dumped on the fill was sluiced with water to wash the fines and chips into the voids of the larger rock. The sluicing plant provided 18 cubic feet of water per second under 275 feet head at the monitors. The ratio of water to rock-fill exceeded 4, which undoubtedly reduced the settlement which otherwise would have taken place. (Figure 7.)

The character of the rock and the methods of shooting produced quarry-run rock varying in size from 10 tons to dust. Normal quarry operation resulted in some trucks being loaded only with large rock, some loaded with small rock, and some loaded with a mixture of intermediate sizes. Selective dumping permitted placing most of the large rock in the downstream section of the fill and most of the small rock in the upstream section of the fill. This was accomplished without interfering with the quarry operation. Mention should be made of the compacting effect on the fill when dumping at the top of a high lift, particularly where the rock was large. It had been intended to smooth up the downstream slope but it was found impossible to move any of the rock without shooting. It was also found to be difficult and expensive to smooth up the slope by adding rock after completion of the fill. For these reasons the downstream slope of the fill was left exactly as placed. Some hand chinking was done on the upstream slope, particularly near the bottom where the voids were large. The surface of each lift was thoroughly scarified and sluiced to remove fines. (Figure 8.)

All filter materials were processed from small quarry rock and tunnel muck. Processed tunnel muck produced an excellent sand. Rigid laboratory control was exercised over the manufacture of the graded filter materials. Besides the usual grading, permeability and density tests, a high head permeability test was used to determine the suitability of the various filter materials for use in the dam. The pyralin walls of the permeameter permitted visual inspection of the filter materials for evidence of piping during testing. The minus 1/2 inch sand required the most careful control and a large share of the tests were confined to this material. The constant head

Figure 7

type of test was used under gradients as high as 104. Besides visual inspection, which for several materials was test enough, a comparison of the grain size distribution before and after the test was made to study the extent of readjustments in the grading of the sand during the test. Records indicate that this constant control over processed filter material resulted in the construction of an effective transition from the fine soil particles to the large quarry-run rock.

The 3 to 10 inch filter material was first placed from the bottom of the dam just several feet ahead of the other filter materials, but, as considerable interference developed in placement, the balance of this material was dumped down for heights of approximately 100 feet from the main rock-fill without undue segregation. The 1/2 to 3 inch and the minus 1/2 inch materials were placed from the bottom using 5-yard dumptors. The sand filter was always placed at a level slightly below the 1/2 to 3 inch rock to preserve continuity of the filters. Both materials were sluiced, spread and compacted with dozers and rollers. As it was necessary to cross the rolled earth-fill in placing these filter materials, considerable care had to be exercised to prevent tracking the earth onto the filter materials and thereby reducing their effectiveness. These filter materials could also be placed to advantage by dumping from the top of the main rock-fill through chutes.

Figure 8

Before any rolled earth-fill was placed, the bottom of the cut-off trench was scrubbed clean with compressed air and water. One borrow pit provided all earth material incorporated in the dam. This was developed as a scraper pit using four carry-all scrapers of from 16 to 20 cubic yards capacity hauled by RD-8 tractors. The earth-fill was placed in the core using bull-dozers to grade and spread the material. Compaction was attained with two sheep's foot rollers mounted side by side, with 176 tamping feet, each having an area of 5.4 square inches and developing a pressure of 293 pounds per square inch when fully loaded. In areas where the roller was inaccessible, the fill was compacted by power tampers. Additional compaction was obtained by the hauling and placing equipment which was not permitted to track. The fill was placed in 6-inch horizontal layers just slightly behind the placement of the downstream filter materials.

For purposes of control and before placing any earth-fill, a test fill was constructed to study compaction of the soil under various combinations of rolling equipment. A study of the test fill results was the basis of the placement criteria developed for the dam fill. Regular periodic sampling was started and laboratory tests performed to check the results against the criteria.

Originally it was planned to require the impervious core to be compacted to a density of 98 pounds per cubic foot dry weight as determined by the Proctor test. At this density, the optimum water content averaged 22.3 per cent. After rolled fill operations were started it was evident that strict adherence to the optimum water content would be impossible. The natural water content of the borrow pit averaged 5 per cent higher than the optimum value and there was no opportunity to dry it because of the excessive rainfall in the area.

A study was made to determine how rapidly the core would consolidate, using soil characteristics obtained from the consolidation test, and assuming the material was placed at a moisture content greater than the optimum value. These studies indicated that by the time construction was completed the earth core would be fully consolidated under the superimposed loads of the upstream rock-fill and filters, no matter if the material was placed wet. With this in mind, the following criteria were developed as compaction requirements for the impervious rolled fill:

1) The minimum compaction should be that which is required by the static state (no drawdown). Under this condition, the required average unit dry density was 94.0 pounds per cubic foot. Dry weights less than 90.0 pounds per cubic foot were limited to one per cent of the samples tested.

2) The minimum water content was limited to 2 per cent less than the optimum value as determined from the Proctor test.

3) The maximum water content was limited by the ability of the hauling equipment to operate without miring.

Placing the material on the wet side of the optimum water content had the advantages of low air voids and a flexible core which could readily adjust itself to the initial readjustments that take place in the main rockfill.

Three methods of control sampling were used on the impervious core:

1. Moisture samples were taken every two hours during the placement of fill to check the water content.

2. Control cylinders were taken at regular intervals to check the density of the fill against that required by the compaction criteria. These cylinders were 5 inches in diameter and 5 inches high, and were taken with a piece of thin walled pipe with a sharpened end. Samples were usually taken at 1-1/2 and 3 feet depths below the surface to avoid disturbance by the sheep's foot roller and hauling equipment. Since the cylinders were calibrated as to weight and volume, the density of the sample was readily determined by weighing the full cylinder and determining the moisture content.

3. Chunk samples were taken at regular intervals to check all physical properties of the fill. This type of sampling required large test pits and interferred somewhat with placement operations. For this reason, tests of this type were held to a minimum. It was the most accurate method of sampling and was therefore used for record testing. The chunks were approximately one cubic foot in size and were taken from the fill at a depth of 3 feet and sometimes 5 feet to obviate any disturbance due to hauling and rolling equipment.

Routine control sampling and testing was performed continuously with the placement of the impervious rolled fill. Moisture content results were generally available within two hours but, where needed, results were obtained in 20 minutes if a volumetric method was used. One control cylinder was taken on the average with every 1200 cubic yards of earth placed. One chunk sample was taken on the average with every 2800 cubic yards of earth placed.

Chunk samples were subjected to a mass gravity test for density determination, specific gravity test, permeability test, mechanical analysis, shear and consolidation tests. Figures 9 to 13 indicate the average results of such tests. Bag samples were also taken adjacent to each cylinder and chunk sample and these were given a compaction test for comparative purposes in studying the degree of compaction given the rolled earth-fill.

The density results of all chunk samples have been plotted in graph form, including wet and dry unit weights, water content, void ratio and per cent air voids. (Figure 14.) Placement of the rolled earth-fill began in June 1941 and was completed in November 1941.

The upstream filters were placed along with the impervious rolled earth core. A stability study of the upstream slope indicated that it was permissible to let the placement of the upstream rock-fill lag as much as 50 feet behind the placement of the rolled earth-fill without any possibility of an earth slide. Rock-fill operations were therefore concentrated on completing the main downstream rock-fill so as not to hold up the rolled earth-fill operations. After the downstream rock-fill had gained considerable lead over the rest of the operations, the upstream rock-fill was started. The upstream rock was dumped in shallow lifts of 10 to 15 feet high, using two dozers to bring the rock to the design slope of 1 on 2-1/2. Most of the rock going into this zone of the dam came from a low level in the spillway quarry and was relatively clean of dirt and dust. Sluicing was done with a monitor fed by a 1500 gallon per minute pump.

Because of the limited working space at the top of the dam, the various zones of material in the upper 40 feet of the embankment were brought up to the crest elevation simultaneously, but the same methods and care in placement were employed. The dam was completed in February 1942, 11 months after the site had been unwatered. The following quantities of material were used:

Quarry-run rock	1,692,000 cubic yards.
Process rock	350,000 cubic yards.
Rolled earth-fill	223,000 cubic yards.
Total quantity measured in the structure	2,265,000 cubic yards.

A view of the completed dam, with the water surface at Elevation 2858, taken October 2, 1942, is shown in Figure 15.

Performance

The Nantahala dam has been in operation nine years. The reservoir has been substantially filled and emptied each season. Careful inspection has failed to disclose any evidence of local settlement or the displacement of rock. The spillway has been used three times.

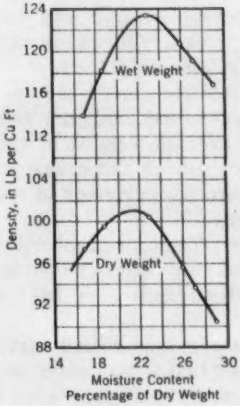


Figure 9

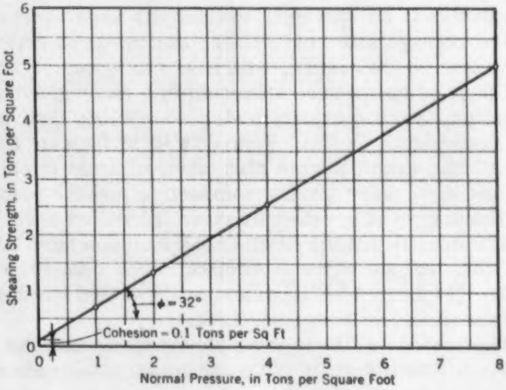


Figure 10

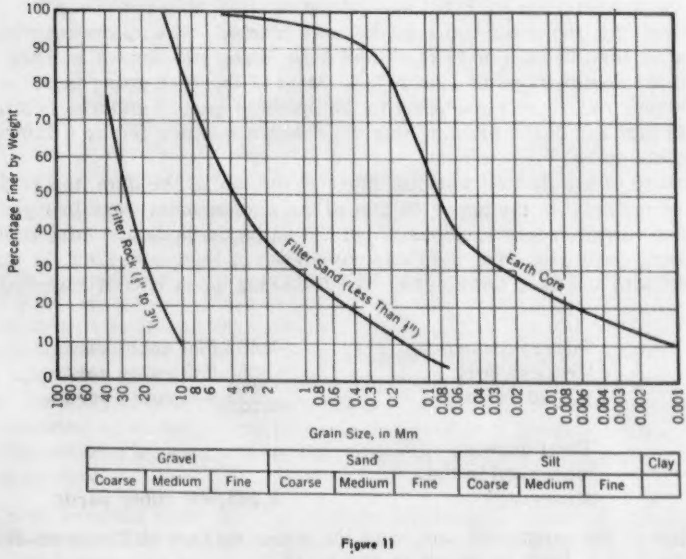


Figure 11

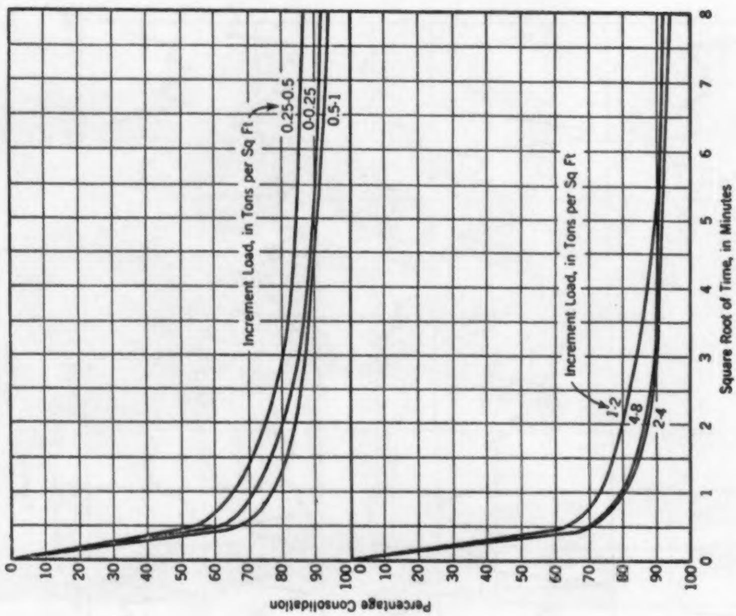


Figure 13

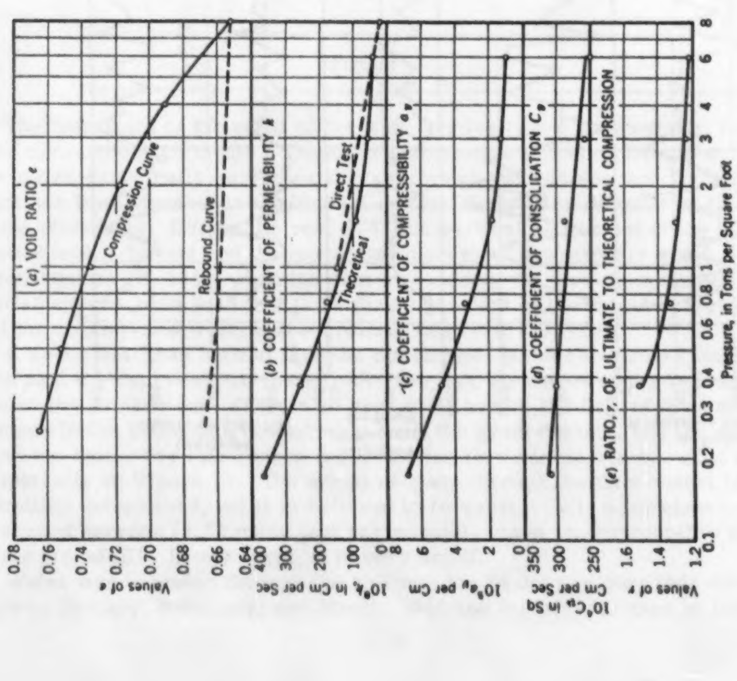


Figure 12

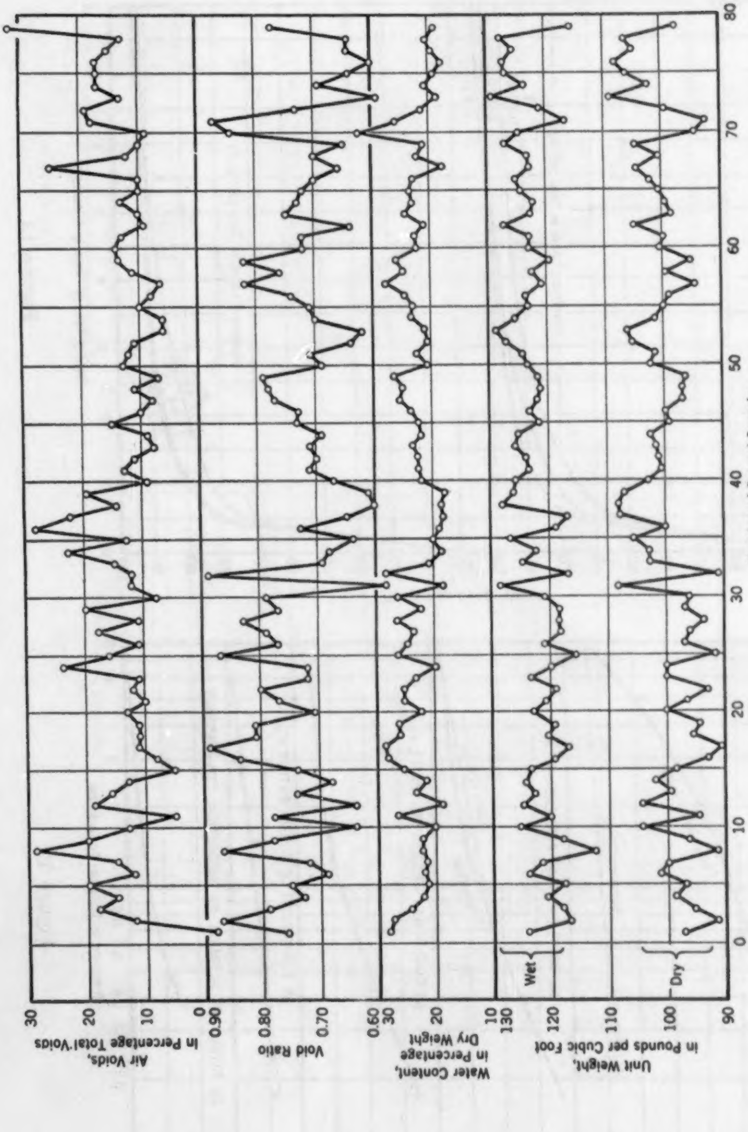

Air Voids,
In Percentage Total Voids
Void Ratio
Water Content,
In Percentage
Dry Weight
Unit Weight,
In Pounds per Cubic Foot
Wet
Dry
Number of Record Sample

Figure 14

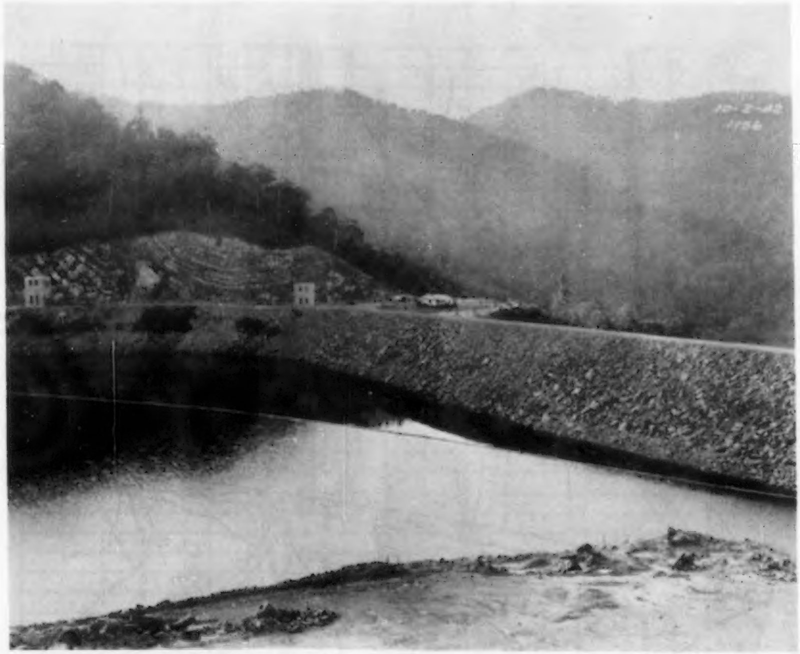
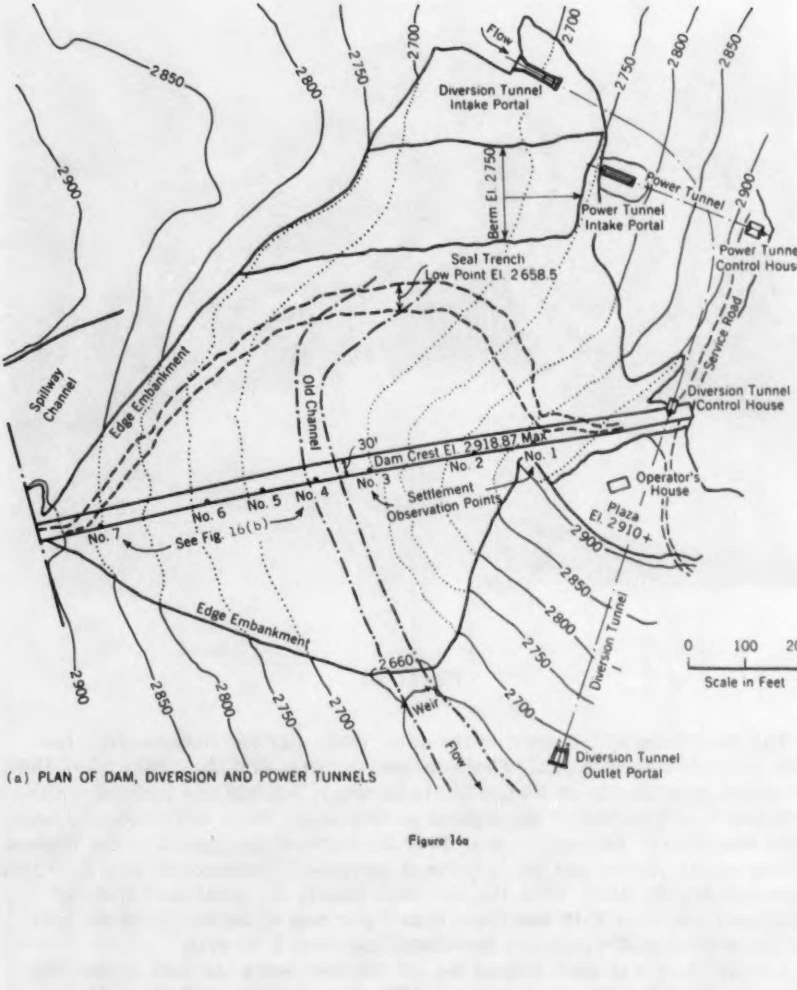


Figure 15

The movement of the crest of the dam, vertically and horizontally, has been measured each month. These measurements, including December 1946, are shown graphically on Figure 16, from which you can see that the settlement has been greatest in the highest section of the dam, decreasing to zero at the abutments. During the year 1947 the vertical settlement at the highest section was 0.19 feet and the horizontal movement downstream was 0.10 feet. Since January 30, 1942, when the dam was closed, the maximum vertical settlement has been 2.16 feet (less than 1 per cent of the height of the fill) and the maximum downstream movement has been 1.01 feet.

A small weir was built across the old channel below the dam so that the flow past the dam could be measured. The flow measured at the weir includes the surface run-off from an area of 12 acres, the flow of two small springs in the abutments downstream from the grout curtain, and the seepage from the dam. For this reason the measured flow varied widely. It is shown graphically on Figure 17. The actual seepage through the dam cannot be definitely determined, but it is believed to be substantially equivalent to the computed seepage (2.75 cubic feet per minute), based on permeability tests of the rolled fill. In any event, it is very small.

Water was released through the spillway for 14 days in May 1944 for 55 days in January, February, and March, 1946 and for several days in 1950.



(a) PLAN OF DAM, DIVERSION AND POWER TUNNELS

Figure 16a

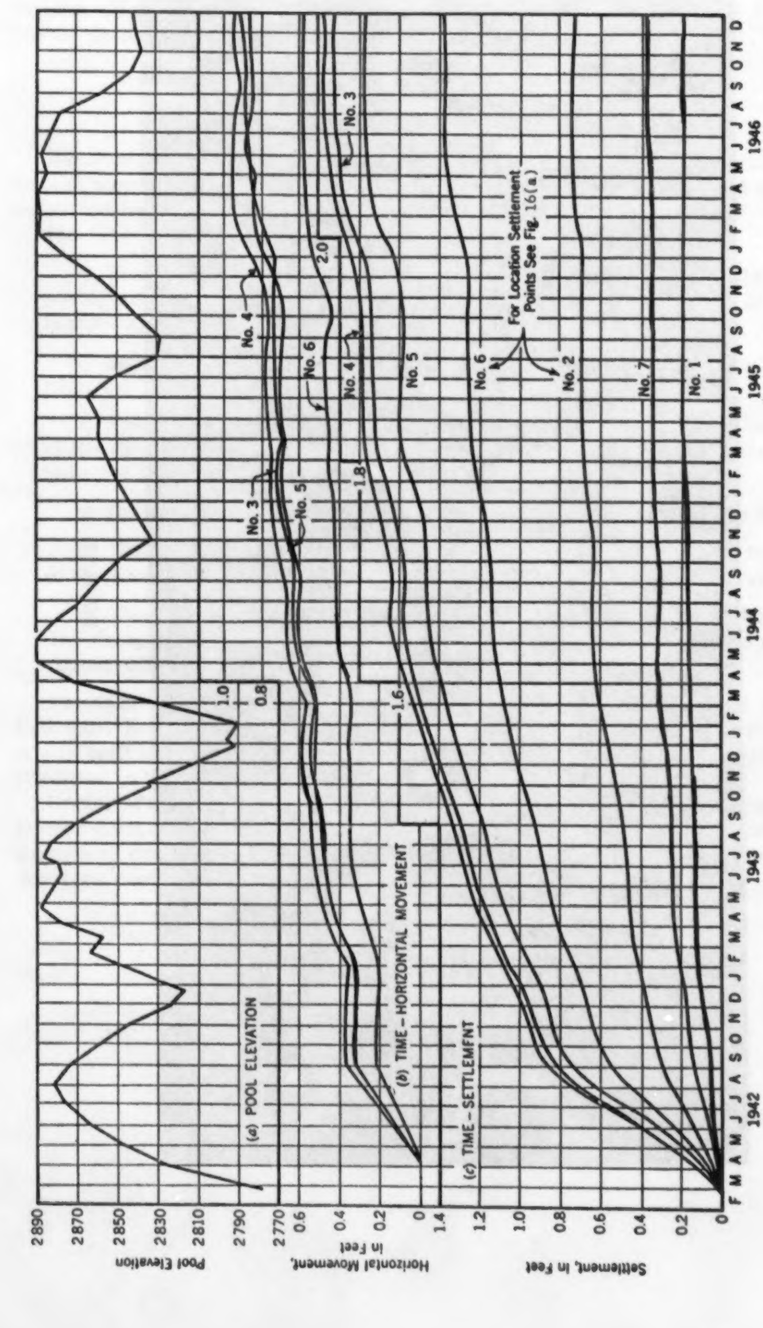

Pool Elevation
Horizontal Movement, in Feet
Settlement, in Feet
(a) POOL ELEVATION
(b) TIME – HORIZONTAL MOVEMENT
(c) TIME – SETTLEMENT
For Location Settlement Points See Fig. 16(a)
No. 1
No. 2
No. 3
No. 4
No. 5
No. 6
No. 7
1942
1943
1944
1945
1946

Figure 16b

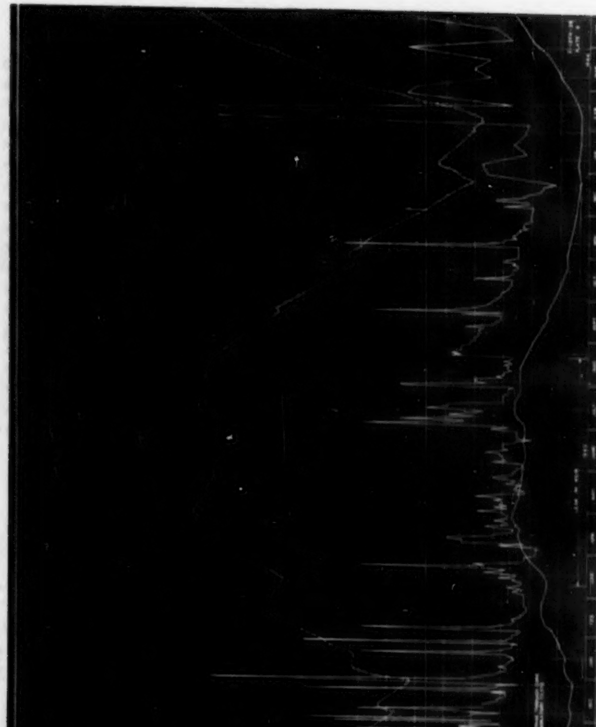


Figure 17

The maximum rate of release was 5,000 c.f.s. The water released through the spillway washed a substantial quantity of loose material from the hillside into the old river channel, as it was expected to do. The debris made a second dam which submerged the measuring weir.

CONCLUSION

The Nantahala rock-fill dam with an impervious earth section on the upstream slope as designed and built, has functioned satisfactorily, with every indication that it will continue to do so for an indefinitely long time.

The following comments appear to be warranted:

1) The materials from which the dam is built will continue to disintegrate, but at a very slow rate, measured in geological rather than in historical time.

2) Settlement of the rock-fill will continue indefinitely at a decreasing rate.

3) The water load is applied normal to the inclined impervious section of the dam. All loads are transmitted directly to the foundation; the heaviest load through the shortest distance.

4) No hydrostatic pressure can be built up in the interior of the dam.

5) The structure is substantially water-tight. The impervious section of the dam adjusts itself to settlement in the rock-fill without rupture.

6) The dam was constructed of materials readily available at the site. The design facilitated construction and no serious difficulties were encountered.

The dam was designed by and constructed under the supervision of the Hydraulic Engineering Department of the Aluminum Company of America. It was built by the Utah Construction Company within the time originally specified in the contract.

Doctor Warren J. Mead, Mr. Joel D. Justin, Mr. William P. Creager, Doctor Glennon Gilboy, and Mr. Robert R. Philippe served as Consulting Engineers during the design and construction of the entire project. Their services contributed immeasurably to its success.

[illegible]

Paper 1743 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: DAMS WITH SLOPING EARTH CORES[a]

James P. Growdon,[1] M. ASCE
(Proc. Paper 1743)

ABSTRACT

This paper briefly reviews the principles which govern the design of a rockfill dam and discusses six rockfill dams with which the author has been intimately acquainted since completion of the Nantahala Dam. Bear Creek Dam, one of these six, incorporates most improvements in design and construction learned from experience.

The conservation and control of water is vital in many parts of the world and is becoming increasingly important in all places where resources are being developed and population increased. Most projects for the control and use of water include a dam at some place in their progress schedule. During the last 30 years a great many dams of widely varying types have been constructed. Particularly in the United States, most of the better dam sites have been occupied leaving the less desirable sites for present and future development. The high cost of building dams has resulted in a search for better and more economical types. As a result of this search the rockfill dam with sloping impervious earth core was developed. The first dam of this type to be constructed was described in a paper entitled "Nantahala Rock Fill Dam with Sloping Impervious Core" presented at the annual meeting of the ASCE in 1948. This paper presented in detail the concept, design, construction, and performance of the Nantahala Dam.

The economy and performance of the Nantahala Dam has been so satisfactory that several similar dams have been built and others are being designed and constructed. This paper is limited to a brief review of the principles which govern the design of a rock fill dam and a discussion of the six rock fill dams with which the writer has been intimately acquainted since the

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1743 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

a. Presented at the October 1956 ASCE Conv. in Pittsburgh, Pa.
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completion of the Nantahala Dam. The major part of the discussion will concern the Bear Creek Dam which was completed in 1953 since it incorporates most of the improvements in the design and the construction of rock fill dams which have been learned from experience.

The rock fill dam should be constructed only on a suitable site. The first requirement is a supply of sound rock adequate for the structure. Sound rock may be described as rock which will not disintegrate in the quarry nor in the handling; which is strong enough to sustain the weight of the dam and the water load and which will not disintegrate rapidly when exposed to weather. The second requirement is an adequate supply of earth suitable for the impervious core. Characteristics of "suitable earth" may vary widely without effecting the usability for an impervious core provided its characteristics are taken into account in the design. If the site permits the use of a diversion tunnel as a pressure conduit after the project has been completed and if the quarry can provide an adequate spillway the economy of the project will be enhanced.

Design

The design of a rock fill dam with sloping earth core is quite simple. It consists of the basic triangular rock section dumped at the natural angle of repose, with an impervious earth core on the upstream slope from which its separated by a suitable filter to prevent the migration of the smaller particles into the voids below. The upstream surface of the earth core is protected by a filter and a layer of rock thick enough to protect the core from wave action and to prevent it from sloughing under sudden draw down.

With one exception dams of this type so far constructed have had rock foundation from which the overburden has been removed. Any foundation which can be described as rock or shale is entirely adequate for even a high rock fill dam. Alluvial material filling a river channel also provides an adequate foundation. The foundation problem is greatly simplified because there is no sharply differential loading on the foundation. Water load is transmitted through the rock fill to the foundation normal to the earth core, so that the heaviest water load reaches the foundation through the shortest path.

A seal trench should be excavated to groutable rock where the earth core is in contact with the foundation. This seal trench should be grouted to whatever degree is necessary to limit the passage of water through the foundation to an acceptable amount. The core is placed directly on the rock in the seal trench.

Characteristics of the earth core can readily be determined in a soils mechanic laboratory and the impervious core designed to suit the characteristics of the material available. The thickness of the core should be such as to limit the passage of water through the dam to an acceptable amount. It should be wide enough to permit placing with mechanical equipment and thick enough to prevent rupture due to differential settlement of the basic rock fill. The lower and upper filter zones can be made of sand and gravel or crushed rock screened and sized so that "The 15% size of the filter shall be larger than 5 times the 15% size and smaller than 5 times the 85% size of the adjacent filter zone. The 15% size and the 85% size as used in this specification are sieve sizes which pass 15% and 85%, respectively, of the material". The upstream rock blanket is designed to prevent the sloughing of the core under conditions of a sudden draw down. The water impounded by the dam compresses the earth core against the filter material below it so that a core of

moderate thickness contains very little water and its angle of internal friction is much greater than it would be for the same material in saturated condition.

All rock dams settle due to the break down of the point contact between the rocks comprising the basic rock fill and because fine material which may separate, some of the larger rocks will in time migrate into the voids below. Settlement is both downward and downstream. The downstream settlement being generally 1/2 magnitude of the vertical. The basic rock fill should be designed convex upstream so that when settlement occurs it will further compact the earth core. The dam should be given a camber so that the center of the structure will be at least as high as the abutments after a long period of settlement.

Existing Dams

Fig. 1 shows the profile of the existing rock fill dams referred to herein. On this you can see that the dams vary in height from 76 feet to 317 feet. With one exception the downstream slope is the natural angle of repose which is substantially one vertical on 1.3 horizontal. The downstream slope of the Kenney Dam consists of a series of berms which brings the average slope to one vertical on 1.75 horizontal. In each of the dams shown the upstream cofferdam has been incorporated in the structure. The upstream slope of the 7 days shown varies from one vertical on 2 horizontal to one vertical on 2.5 horizontal with a steeper slope at the top of all except Kenney Dam.

The following are pictures of several of the dams under construction and completed.

Figure 2	Queens Creek Dam - Reservoir Filled
Figure 3	Queens Creek - Spillway Channel
Figure 4	Queens Creek - Fuse Plug Controlling Spillway
Figure 5	Nantahala Dam - Downstream Face
Figure 6	Nantahala Dam - Upstream Face with Partially Filled Dam
Figure 7	Nantahala - Flood Gates and Fuse Plug
Figure 8	Cedar Cliff Dam - Under Construction
Figure 9	Cedar Cliff Dam - Partially Filled Reservoir
Figure 10	Cedar Cliff Dam - Cascade Spillway Discharging 3,000 c.f.s.
Figure 11	Bear Creek Dam - Under Construction
Figure 12	Bear Creek Dam - Under Construction
Figure 13	Bear Creek Dam - Downstream Face
Figure 14	Bear Creek Dam - Full Reservoir
Figure 15	Bear Creek Dam - Constructing Fuse Plug
Figure 16	Bear Creek Dam - Cascade Spillway
Figure 17	Bear Creek Dam - Cascade Spillway Discharging 2,000 c.f.s.
Figure 18	Tennessee Creek Dam - Partially Filled Reservoir
Figure 19	Tennessee Creek Dam - Tainter Gate and Fuse Plug Controlling Spillway
Figure 20	Wolf Creek Dam - Partially Filled Reservoir
Figure 21	Wolf Creek Dam - Tainter Gate and Fuse Plug

The rock used in Nantahala was very hard massive arkose. Quarrying methods produced rock of all sizes. Some single rocks were a full load for a truck. The rock at Queens Creek was also arkose but much less massive. Rock at Cedar Cliff, Bear Creek, Wolf Creek, and East Fork is a good quality shist much softer than the arkose. The rock used for the Kenney Dam was very sound hard basalt.

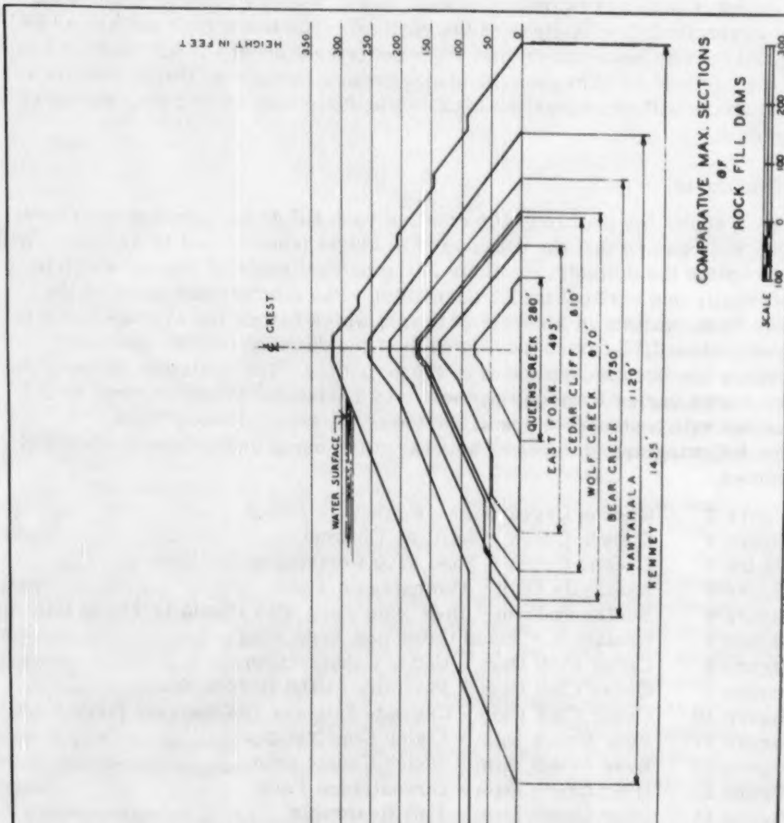
HEIGHT IN FEET
350
300
250
200
150
100
50
0
CREST
WATER SURFACE
QUEENS CREEK 280'
EAST FORK 495'
CEDAR CLIFF 610'
WOLF CREEK 670'
BEAR CREEK 750'
NANTAHALA 1120'
KENNEY 1455'
COMPARATIVE MAX. SECTIONS
OF
ROCK FILL DAMS
SCALE 100 0 100 200 300

Figure 1

Figure 2

Figure 3

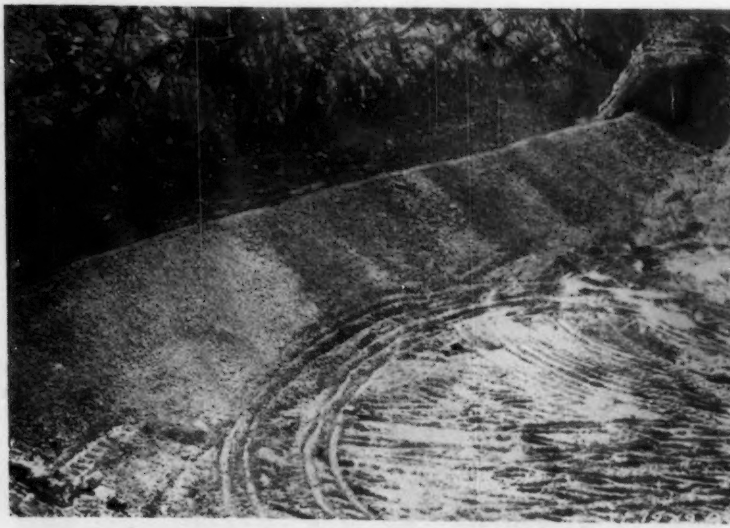

Figure 4

Figure 5

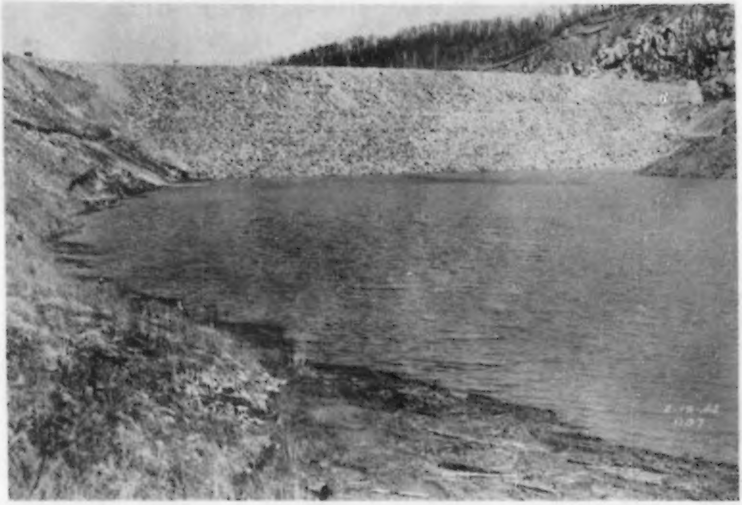

Figure 6

Figure 7

Figure 8

Figure 9

Figure 10

Figure 11

Figure 12

Figure 13

Figure 14

Figure 15

Figure 16

Figure 17

Figure 18

Figure 19

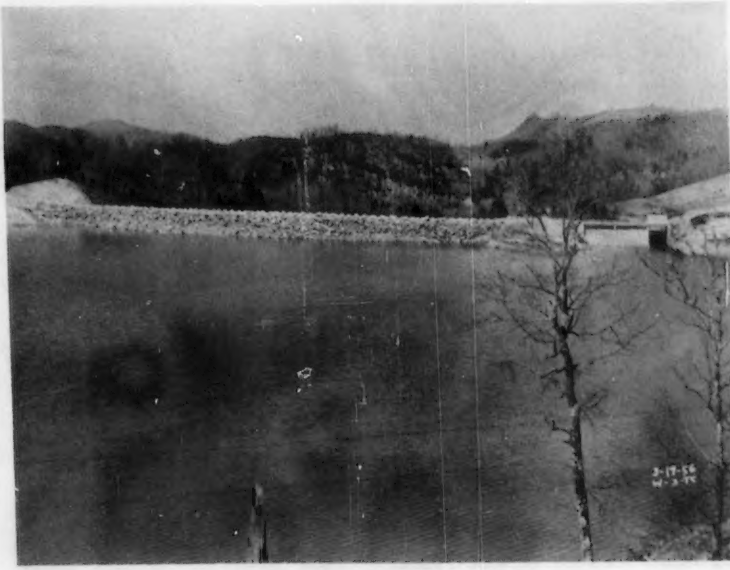

Figure 20

Figure 21

Unlimited quantities of sand, gravel, and silt were available at the Kenney site. Sand and gravel was used for the filter layers and the silt for the impervious core. The impervious earth available at Nantahala, Queens Creek, Cedar Cliff, Bear Creek, Wolf Creek, and East Fork dams was quite similar although by no means identical. Fig. 22 shows the optimum soil conditions, Fig. 23 shows the in place soil conditions, and Fig. 24 shows the general soil characteristics for each of these six dams. Fig. 24 also shows the vertical and horizontal settlement and the leakage.

You will note that the settlement varies from 2.85 feet vertical, 1.53 feet horizontal to 0.28 feet vertical and 0.18 feet horizontal at Queens Creek. The amount of settlement is partly determined by the construction procedure.

Leakage

There has been no measurable leakage passed the Kenney Dam. In case of the other dams leakage has been collected immediately below the dam and measured when the reservoir was full during dry weather. The leakage varies from 14.6 cubic feet per minute at Wolf Creek to 2.67 cubic feet per minute at East Fork. It is probable that most or all of the measured leakage is through the foundation rather than through the impervious core. The small amount of water which passes through the core would certainly be evaporated before reaching the point where it can be measured.

Bear Creek Dam

Experience gained in constructing these several rock rill dams has resulted in improving some features of the design and the construction procedures used at Nantahala. The Bear Creek Dam completed in November 1953 incorporates most of these improvements. A more detailed description of the design and construction of Bear Creek will illustrate the latest procedures for a rock fill dam at the Bear Creek site. In this connection it should be emphasized that no two dam sites are identical and the design and construction procedures should be varied to suit the particular site.

The Site

The Bear Creek site is on the East Fork of the Tuckasegee River in western North Carolina. The drainage area is 75.3 square miles. The average flow is 207 c.f.s. The maximum flood on record is 21,000 c.f.s. and the minimum flow 1.0 c.f.s. The site is a narrow gorge with massive shist on the right abutment and somewhat less massive shist on the bottom and left abutment. The site would be suitable for a concrete gravity dam but would require extensive excavation to reach a suitable foundation particularly on the left abutment. A supply of earth adequate for an earth dam is not readily available. It is an excellent site for a rock fill dam.

Fig. 25 is a plan of the entire project. The dam is 215 feet high and 775 feet long at the crest. It contains 850,000 cubic yards of rock, 70,000 cubic yards of crushed rock filter and 165,000 cubic yards of compacted earth measured in the dam. The dam creates a reservoir with a surface area of 476 acres and a useful storage of 4,536 acre feet.

Design

The basic design of the Bear Creek project consists of a rock fill dam with an unlined diversion tunnel 1,484 feet long, through the right abutment. The

SUMMARY OF ALCOA DAMS SOIL DATA

	Nantahala	Queens Creek	Cedar Cliff	Bear Creek	Wolf Creek	East Fork	Chilhowee
I. Optimum Soil Conditions							
Water Content (percent)							
Average	21.8	22.1	21.4	22.8	22.6	24.5	25.3
Maximum	28.3	24.0	23.8	30.8	30.5	25.2	27.0
Minimum	16.8	20.0	19.5	17.4	19.0	23.8	24.2
Dry Density (lbs./cu.ft.)							
Average	98.2	98.4	99.4	97.6	97.9	95.7	95.8
Maximum	105.8	100.2	101.1	108.5	105.0	98.5	98.0
Minimum	88.2	95.0	97.0	85.9	84.0	92.8	94.2

Figure 22

SUMMARY OF ALCOA DAMS SOIL DATA

	Nantahala	Queens Creek	Cedar Cliff	Bear Creek	Wolf Creek	East Fork	Chilhowee
II. In-Place Soil Conditions							
Water Content (percent)							
Average	22.6	--	24.7	25.2	23.0	24.9	26.7
Maximum	28.8	--	35.0	39.5	36.1	33.1	33.5
Minimum	17.7	--	15.3	15.2	10.0	16.0	21.2
Dry Density (lbs./cu.ft.)							
Average	100.0	--	--	96.0	98.0	98.0	94.3
Maximum	109.0	--	--	102.5	108.8	103.0	104.8
Minimum	90.8	--	--	83.1	83.8	90.0	87.1
Permeability (cm./sec.X 10^{-4})							
Average	0.0187	0.0195	0.2513	0.0260	0.1874	0.0100	0.0049
Maximum	0.0762	0.0430	1.8900	0.1880	1.7000	0.0500	0.1760
Minimum	0.0014	0.0027	0.0084	0.0012	0.0014	0.0015	0.0006
Shear							
Friction Angle (degrees)							
Average	31.4	31.7	32.7	31.8	30.6	30.9	30.5
Maximum	35.0	34.3	36.0	34.3	33.1	33.4	32.0
Minimum	27.9	28.5	29.5	28.8	27.7	28.0	29.0
Cohesion (lbs./sq.ft.)							
Average	340	403	710	375	455	420	850
Maximum	800	600	880	735	680	890	1100
Minimum	0	200	540	0	180	230	700

Figure 23

SUMMARY OF ALCOA DAMS SOIL DATA

	Nantahala	Queens Creek	Cedar Cliff	Bear Creek	Wolf Creek	East Fork	Chilhowee
III. General Soil Characteristics							
Specific Gravity	2.77	2.79	2.76	2.75	2.78	2.78	2.82
Mechanical Analysis							
15% Size (mm.)							
Average	0.0040	0.0011	0.4900	0.0016	0.0015	0.0032	0.0020
Maximum	0.0200	0.0016	0.1300	0.0210	0.0200	0.0150	0.0080
Minimum	0.0001	0.0010	0.0010	0.0007	0.0001	0.0009	0.0002
85% Size (mm.)							
Average	0.343	0.466	1.950	0.412	0.390	0.432	0.200
Maximum	0.950	1.800	4.000	1.520	1.900	2.500	0.600
Minimum	0.170	0.036	0.050	0.059	0.040	0.110	0.150
IV. Vertical Settlement (ft.)	2.85	0.28	1.08	0.95	0.44	0.43	0.16
Vertical Settlement (% of height)	1.12	0.36	0.65	0.43	0.27	0.32	0.21
V. Horizontal Movement (ft.)	1.53	0.18	0.26	0.54	0.28	0.16	0.12
VI. Years of Settlement Observation	15.2	8.3	5.5	3.8	2.8	2.8	0.8
VII. Leakage (c.f.m.)	4.74	3.1	4.3	8.0	14.6	2.67	--

Figure 24

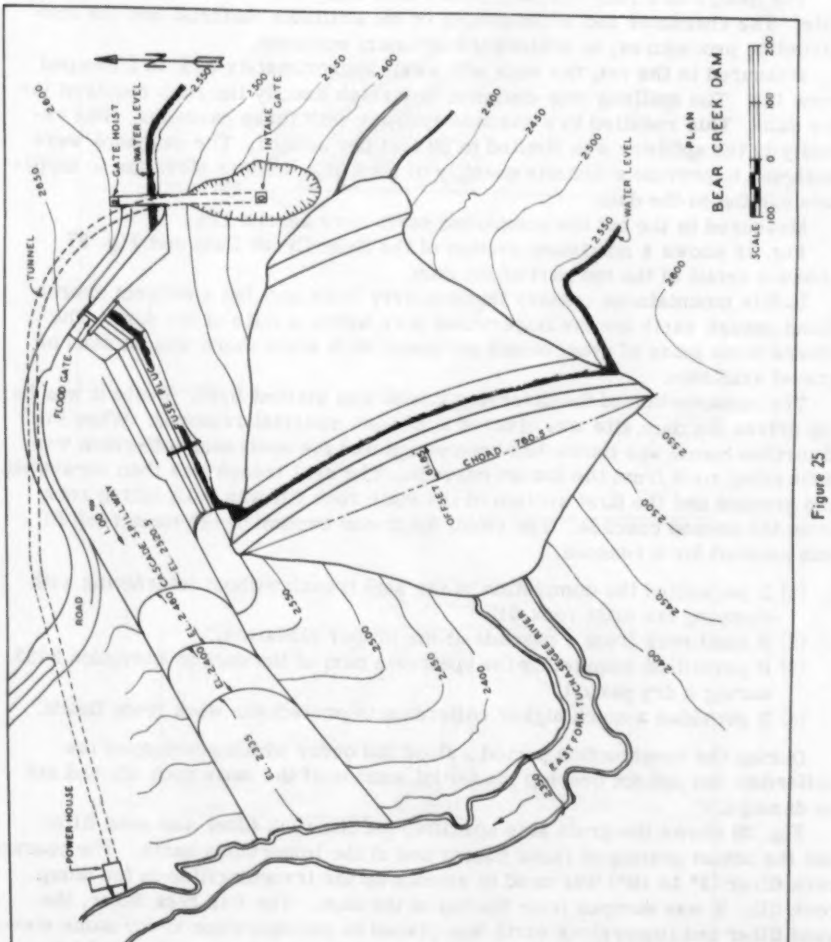
BEAR CREEK DAM
PLAN
SCALE
100
0
100
200
N
GATE HOIST
WATER LEVEL
INTAKE GATE
TUNNEL
FLOOD GATE
FUSE PLUG
CASCADE SPILLWAY
EL. 2520.5
EL. 2 460
EL. 2400
EL. 2325
ROAD
POWER HOUSE
OFFSET 61.85'
CHORD 780.5'
EAST FORK TUCKASEGEE RIVER
2850
2600
2550
2500
2450
2400
2350

Figure 25

lower end of the diversion tunnel to be lined and used as a power conduit. A 10,000 KW hydroelectric unit is installed in a small powerhouse at the exit end of the diversion tunnel. On the right abutment there is a cascade spillway with a capacity of 60,000 c.f.s. controlled by one 25 feet by 25 feet tainter gate and a fuse plug.

The design of a rock fill dam should take into account the conditions of the site. The character and accessibility of the available material and the construction procedures, to achieve the optimum economy.

Measured in the cut, the rock will swell approximately 35% in a dumped rock fill. The spillway was designed to furnish exactly the rock required for the dam. This resulted in a cascade spillway with three cascades. The velocity in the spillway was limited to 20 feet per second. The cascades were designed to provide a definite quantity of rock at a definite elevation to facilitate hauling to the dam.

Measured in the cut the compacted earth core shrank 17%.

Fig. 26 shows a maximum section of the Bear Creek Dam and Fig. 27 shows a detail of the top part of the dam.

In this mountainous country there is very little soil but a diligent search found enough earth for the impervious core within a mile of the dam. The filters were made of crushed and screened rock since there was no sand or gravel available.

The construction of the diversion tunnel was started first. While it was being driven the dam site was cleared and loose material removed. When the diversion tunnel and intake had been completed the upstream cofferdam was built using rock from the lowest cascade. The seal trench was then excavated and grouted and the first section of the main rock fill was built taking rock from the second cascade. The small upstream section of the main rock fill was adopted for 4 reasons.

(1) It permitted the completion of the seal trench without interfering with dumping the main rock fill.
(2) It used rock from a cascade at the proper elevation.
(3) It permitted completing the upstream part of the dam to Elevation 2435 during a dry period.
(4) It provided a much higher cofferdam to protect the work from floods.

During the construction period a flood did occur which overtopped the cofferdam but did not overtop the initial section of the main rock fill and did no damage.

Fig. 28 shows the grain size specified for the rock filter and sand filter and the actual grating of these filters and of the impervious earth. The coarse rock filter (3" to 10") was used to smooth up the irregularities in the dump rock fill. It was dumped from the top of the dam. The fine rock filter, the sand filter and impervious earth was placed in one operation at the same elevation. Approximately one foot thickness of the filter layers migrated into the voids below during the placing operation.

Impervious material was hauled to the site in trucks, spread by bulldozers in 8" layers and compacted with sheep's foot rollers. Care was exercised to be sure that no core was placed at less than the optimum water content. The operation of equipment on the impervious fill set the limit to the maximum amount of water permissible in the core material. The surface of the impervious fill was sloped at all times so that rain water would readily drain off the fill. The upstream filter and the upstream rock blanket are kept to

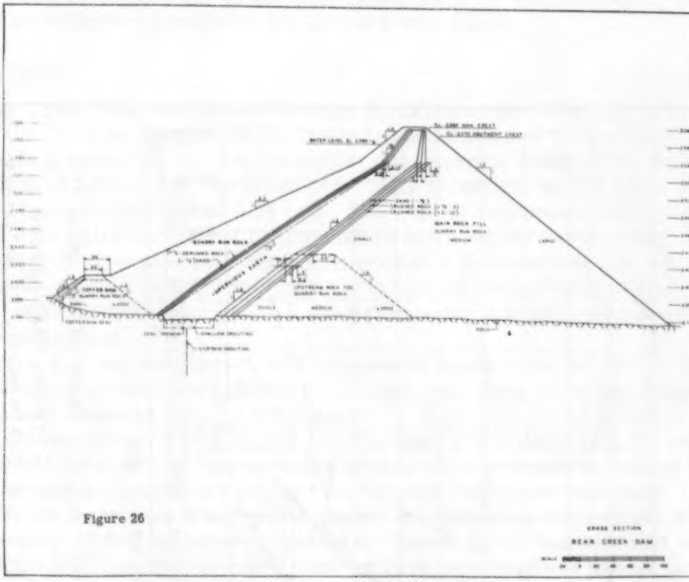

Figure 26

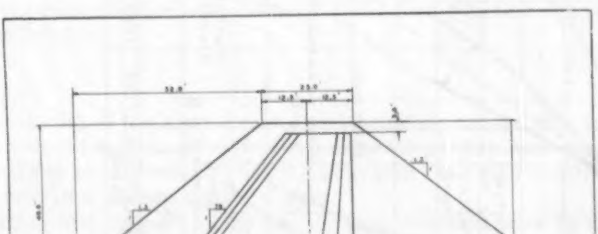

Figure 27

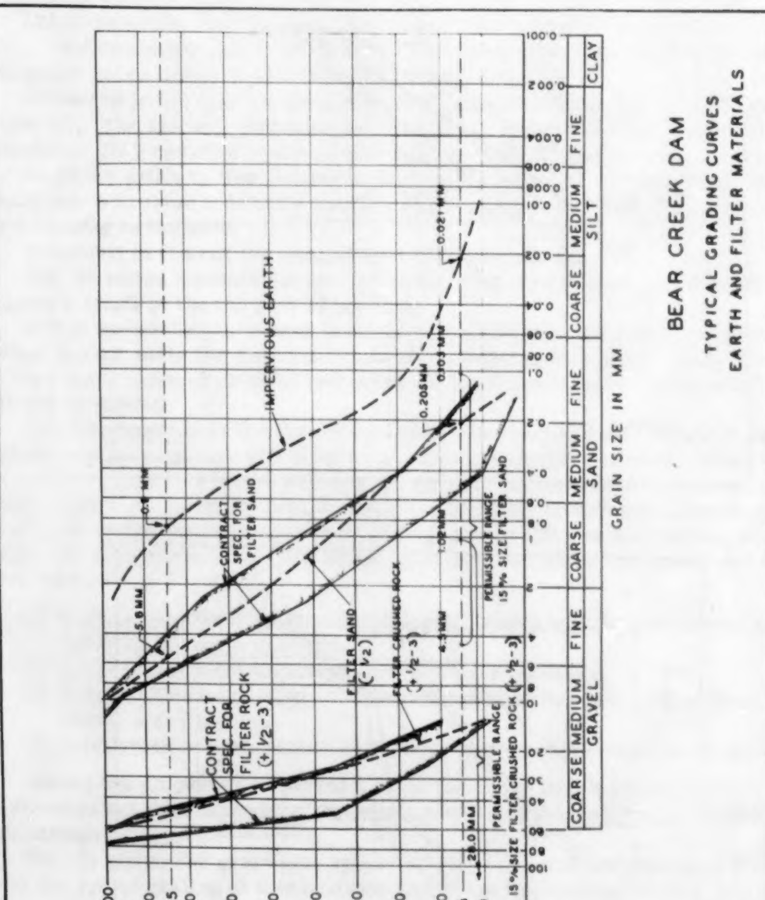
BEAR CREEK DAM
TYPICAL GRADING CURVES
EARTH AND FILTER MATERIALS
% FINER BY WEIGHT
GRAIN SIZE IN MM
COARSE
MEDIUM
FINE
GRAVEL
SAND
SILT
CLAY
IMPERVIOUS EARTH
CONTRACT SPEC. FOR FILTER SAND
FILTER SAND (-1/2)
CONTRACT SPEC. FOR FILTER ROCK (+1/2-3)
FILTER CRUSHED ROCK (+1/2-3)
PERMISSIBLE RANGE 15% SIZE FILTER SAND
PERMISSIBLE RANGE 15% SIZE FILTER CRUSHED ROCK (+1/2-3)
5.8 MM
0.5 MM
0.205 MM
0.021 MM
1.02 MM
4.3 MM
28.0 MM

Figure 28

within a few feet of the top of the impervious fill as it was placed. This prevented washing the impervious fill during heavy rains.

Performance

Bear Creek Dam was closed October 8, 1953 and the reservoir was substantially filled by January 1954. Delay in installation of the hydroelectric unit made it necessary to use the spillway for several weeks discharging a maximum of 5,000 c.f.s. As noted previously a vertical settlement has been 0.95 feet and the horizontal 0.54 feet. The rate of settlement has decreased so that very little settlement is expected to occur in the future. The small settlement of approximately one-half per cent of the height of the dam is primarily due to placing the basic rock fill in one lift rather than in several lifts, as was done at Nantahala: and to a very thorough job of sluicing the rock fill as it was dumped.

The rock in the seal trench was reasonably impervious and only a moderate amount of grouting was done. No attempt was made to be absolutely sure that no water passed through the foundation. A perfect job of grouting would have cost more than it was worth. The leakage of 8.0 cubic feet per minute is very small for a dam of this size and height. It is believed that all of it occurs through the foundation rather than through the impervious core.

Since the reservoir filled on November of 1953 it has been drawn down approximately 25 feet on several occasions. Generally it has operated within 2 or 3 feet of full. The leakage has remained relatively constant except during rainy weather when the water from a substantial drainage area is included in the measured amount. No maintenance of any kind has been required. There is reason to believe that no maintenance of the dam will be required in the foreseeable future.

Economy

The actual cost of the Bear Creek Dam including the spillway has been compared with the estimated cost of the concrete gravity dam creating a reservoir of the same size. A rock fill dam cost less than one-third of the estimated cost of a concrete gravity dam.

Information with respect to the Kenney Dam was furnished by the International Engineering Company of San Francisco, California with the approval of Mr. McNeely DuBose, Vice President of the Aluminum Company of Canada. The International Engineering Company designed the Kemono Hydroelectric Development including the Kenney Dam. The information with respect to the Nantahala, Queens Creek, Cedar Cliff, Bear Creek, Wolf Creek, and East Fork Dams was obtained from the Hydraulic Engineering Department of the Aluminum Company of America with the assistance of Mr. B. J. Fletcher, Chief Hydraulic Engineer and Chief Assistant Engineer of Alcoa. The filter and soil characteristics were supplied by Mr. H. C. Sperry, Engineer, Hydraulic Department, Alcoa who had charge of the soils mechanics investigations for each of the 6 Alcoa Dams.

Paper 1744 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: PERFORMANCE OF SEVEN SLOPING CORE DAMS

James P. Growdon,[1] M. ASCE
(Proc. Paper 1744)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

ABSTRACT

The design and construction of seven dams is briefly reviewed and performance is discussed measured in terms of leakage, settlement, control and passage of floods, operation, repairs and maintenance. Conclusion reached that sloping core rockfill dams so far designed and constructed have performed to entire satisfaction of owners and operators.

The Aluminum Company of America in the period 1940 to 1957, inclusive, has designed and built seven sloping core rockfill dams. These dams are owned and operated by the Nantahala Power & Light Company and Tapoco, Inc., subsidiaries of Alcoa.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1744 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

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The Power Division believes that the performance of these sloping core rockfill dams would be of interest to all members of the Society who are concerned with dam construction; hence this paper.

A very brief description of the design and construction of these dams is a necessary preliminary to a report of their performance.

The basic structural element of each of these dams is a dumped rockfill with slopes the natural angle of repose. The rock for two of the dams is sound, hard arkose. The rock used in five of the dams is schist, somewhat softer than the arkose. In all but one case the excavation of a spillway channel through an abutment provided the material with which to build the dam.

An impervious earth core on the upstream slope of the basic rockfill provides a watertight membrane. This core is protected on both sides by a graded filter which prevent the migration of the finer particles into the larger voids. The core is protected from wave action and from sloughing into the reservoir by a rock blanket which is free draining.

The earth core is in direct contact with the rock in the bottom and the abutments in a seal trench excavated to sound rock and grouted. The thickness of the earth core is determined by three factors:

1. To eliminate the leakage through the core to an acceptable amount (very small).
2. To permit placing with mechanical equipment.
3. To prevent rupture due to possible differential settlement.

The dams are curved upstream and are cambered so that the center section of the dam is built to a higher elevation than the abutments.

The spillway channel through the rock of the abutment is unlined. In the Nantahala Dam the flood water flows down a steep slope into the river bed well below the toe of the dam. In the other dams the unlined rock spillway channel ends in one or more cascades which return the flood water to the original river channel quite close to the toe of the dam. The spillways are designed in accordance with the TVA flood formula and will pass a flood much larger than any flood which can reasonably be expected ever to occur.

With two exceptions, the spillways are controlled by Tainter gates with a capacity equal to the "Hundred Year" flood and by fuse plugs for the remainder of the spillway designed flood. These fuse plugs are small sloping core rockfill dams which will wash out in sections when overtopped.

Nantahala Dam

The Nantahala Dam is the first dam of this type to be constructed. (Figs. 1 and 2) It is located on the Nantahala River in western North Carolina.

Drainage Area	-	91 sq. miles
Height	-	250 ft.
Length	-	1042 ft.
Total Volume	-	2,265,000 cu. yds.
Useful Storage	-	126,000 ac. ft.

Queens Creek Dam

The Queens Creek is the second of the sloping core rockfill dams to be constructed by Alcoa. (Figs. 3 and 4) It is located on Queens Creek in western North Carolina, close to the Nantahala Powerhouse.

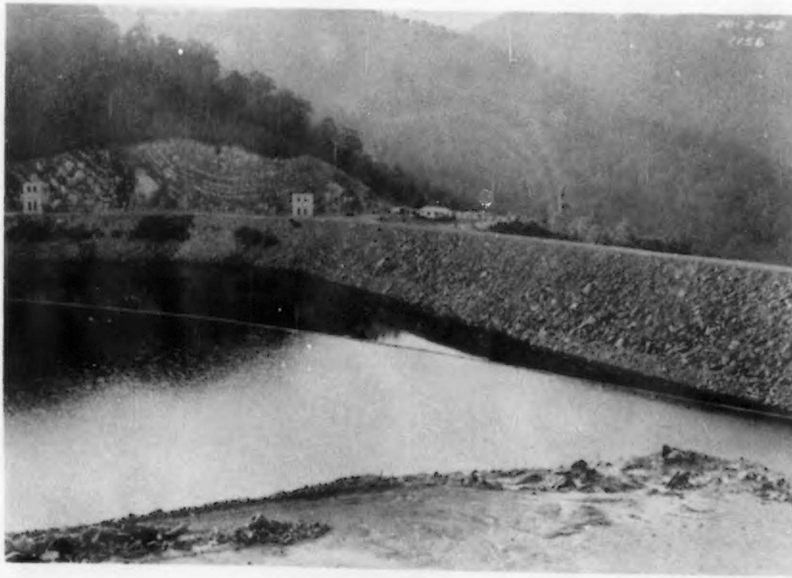


Figure 1

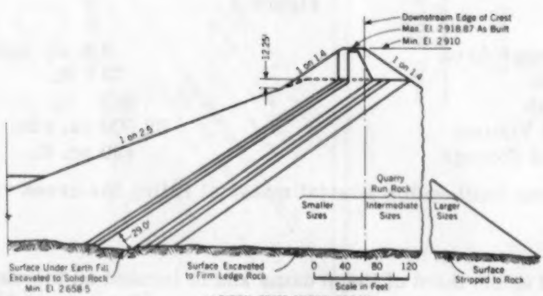


Figure 2

Figure 3

Drainage Area	-	3.6	sq. miles
Height	-	78	ft.
Length	-	382	ft.
Total Volume	-	89,000	cu. yds.
Useful Storage	-	490	ac. ft.

This dam was built on the alluvial material filling the creek valley.

Cedar Cliff

Cedar Cliff is the third of these dams and is located on the east fork of the Tuckasegee River in western North Carolina. (Figs. 5 and 6)

Drainage Area	-	80.7	sq. miles
Height	-	165	ft.
Length	-	600	ft.
Total Volume	-	686,000	cu. yds.
Useful Storage	-	585	ac. ft.

Bear Creek

Bear Creek is the fourth sloping core rockfill dam built by Alcoa. (Figs. 7 and 8) It is located on the east fork of the Tuckasegee River in western North Carolina, at the upper end of the Cedar Cliff reservoir.

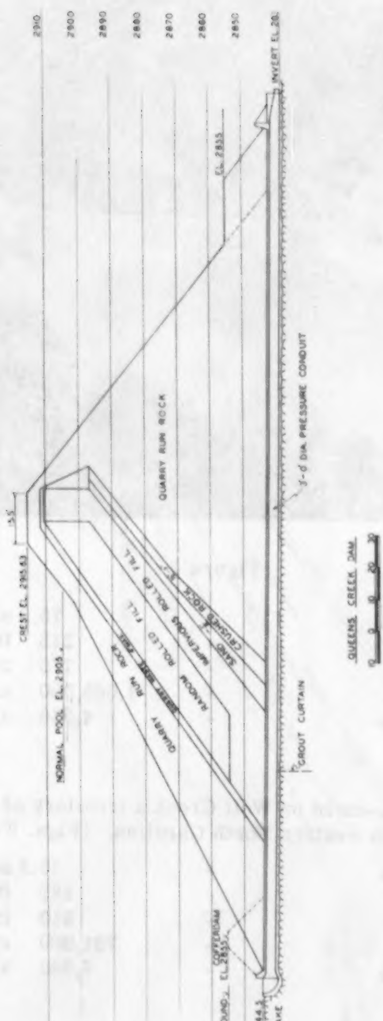

CREST EL. 2915.83
NORMAL POOL EL. 2905
QUARRY RUN ROCK
CRUSHED ROCK
SAND
IMPERVIOUS ROLLED FILL
RANDOM ROLLED FILL
GROUT CURTAIN
PRESSURE CONDUIT
EL. 2855
COFFERDAM
ORIGINAL GROUND
INTAKE
QUEENS CREEK DAM
2910
2900
2890
2880
2870
2860
2850

Figure 4

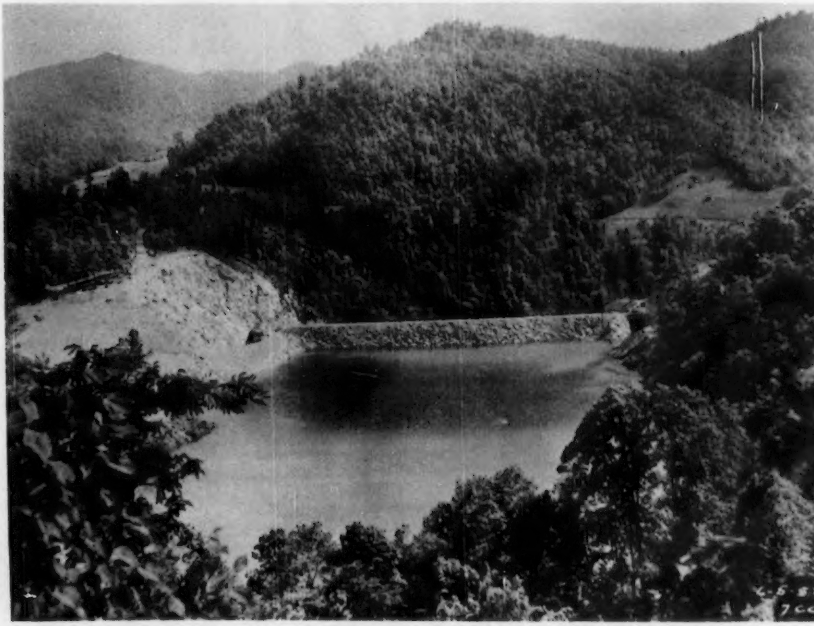


Figure 5

Drainage Area	-	75.3	sq. miles
Height	-	215	ft.
Length	-	775	ft.
Total Volume	-	1,085,000	cu. yds.
Useful Storage	-	4,536	ac. ft.

Wolf Creek

Wolf Creek Dam is located on Wolf Creek a tributary of the east fork of the Tuckasegee River in western North Carolina. (Figs. 9 and 10)

Drainage Area	-	15.2	sq. miles
Height	-	185	ft.
Length	-	810	ft.
Total Volume	-	731,000	cu. yds.
Useful Storage	-	7,640	ac. ft.

East Fork Dam

This dam is on a tributary of the east fork of the Tuckasegee River in western North Carolina. (Figs. 11 and 12)

Drainage Area	-	24.9	sq. miles
Height	-	135	ft.
Length	-	385	ft.

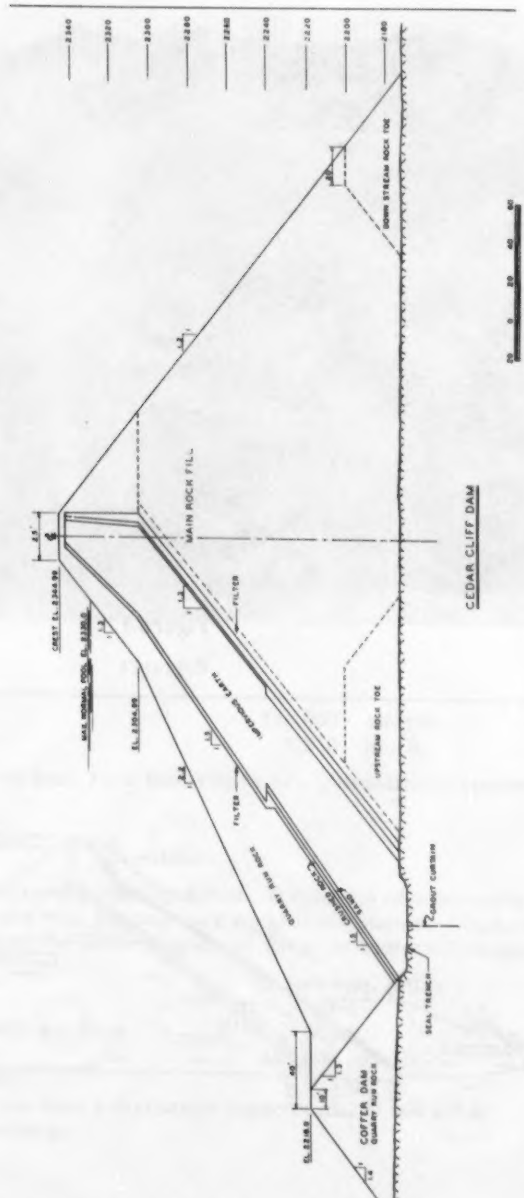

CEDAR CLIFF DAM
CREST EL. 2344.00
MAX. NORMAL POOL EL. 2330.0
EL. 2304.00
EL. 2218.0
MAIN ROCK FILL
IMPERVIOUS EARTH
FILTER
QUARRY RUN ROCK
CRUSHED ROCK
UPSTREAM ROCK TOE
DOWN STREAM ROCK TOE
COFFER DAM
QUARRY RUN ROCK
GROUT CURTAIN
SEAL TRENCH
2340
2320
2300
2280
2260
2240
2220
2200
2180

Figure 6

Figure 7

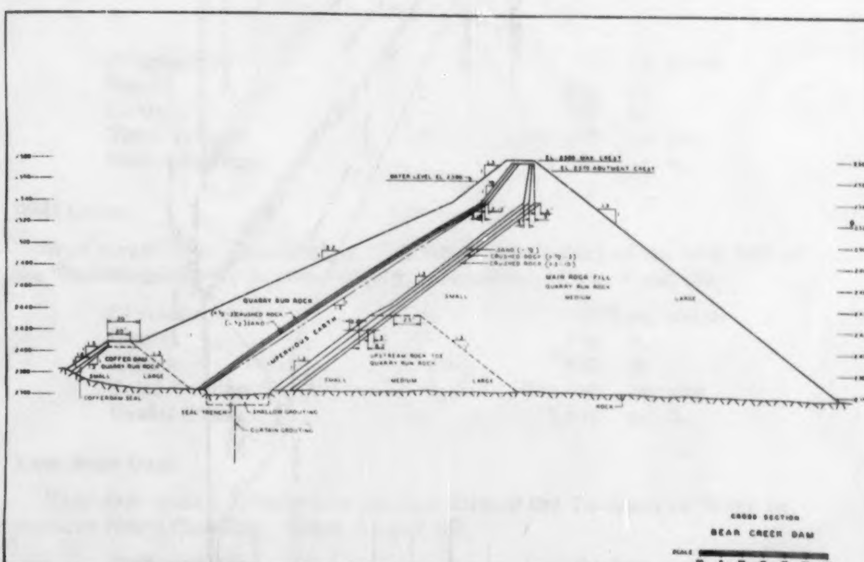


Figure 8

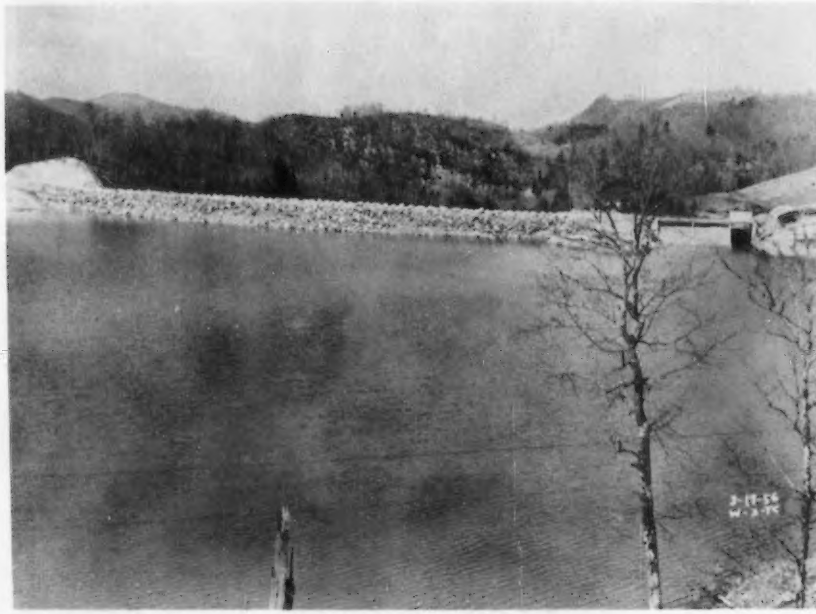


Figure 9

Total Volume	-	195,000	cu. yds.
Useful Storage	-	1,250	ac. ft.

N.B. The Wolf Creek and East Fork Reservoirs are hydraulically connected by a tunnel.

Chilhowee Dam

This is the latest dam constructed by Alcoa. It consists of a concrete gravity spillway and intake with sloping core rockfill abutments. (Figs. 13, 14 and 16) It is located on the Little Tennessee River in eastern Tennessee.

Drainage Area	-	1,977 sq. miles
Height	-	91 ft.
Length of rockfill sections	-	655 ft.
Total Volume	-	353,000 cu. yds.
Useful Storage	-	6,805 ac. ft.

Six Tainter Gates have a discharge capacity of 230,000 c.f.s. with a 5 ft. surcharge.

Performance

The performance of a dam may be measured in terms of leakage, settlement, control and passage of floods, operation, repairs and maintenance.

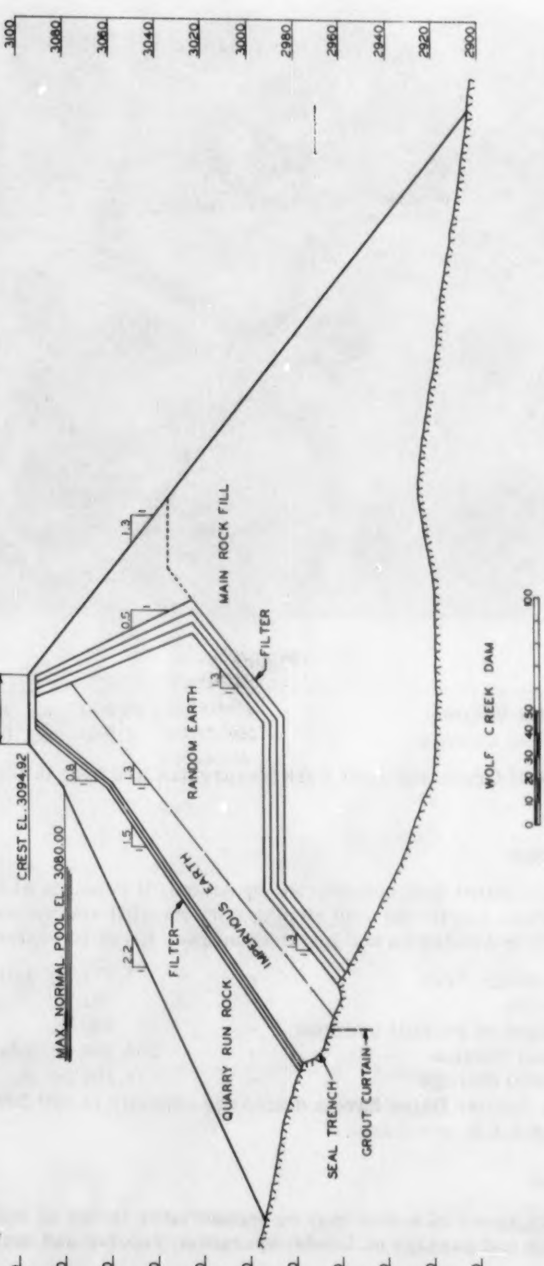

CREST EL. 3094.92
30'
MAX. NORMAL POOL EL. 3080.00
QUARRY RUN ROCK
FILTER
IMPERVIOUS EARTH
RANDOM EARTH
MAIN ROCK FILL
FILTER
SEAL TRENCH
GROUT CURTAIN
WOLF CREEK DAM
2.2
1.5
1.3
0.8
1.3
0.5
1.3
1.3
3100
3080
3060
3040
3020
3000
2980
2960
2940
2920
2900
0 10 20 30 40 50 100

Figure 10

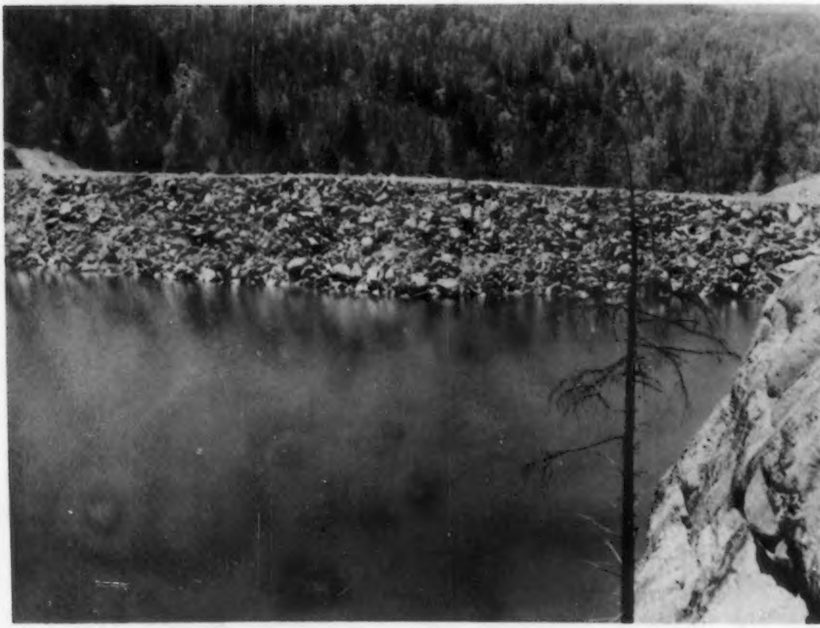


Figure 11

Nantahala

Diversion closed February, 1942—Reservoir filled July, 1942. The reservoir has been substantially emptied and filled each year.

A weir was constructed downstream from the dam to measure leakage. The water measured included the runoff from a 12 acres drainage area and two small springs downstream from the grout curtain in addition to the leakage through the foundation and through the earth core. The water passing the weir was measured frequently until January 1950 at which time the weir was rendered inoperative by flood water discharge through the spillway. With a full reservoir in dry weather the water measured at the weir amounted to 36 gallons per minute. During rains this increased to as much as 135 gallons per minute. There is no evidence that the leakage past the dam has increased since the reservoir was first filled. It is believed that all of the water which passes through the earth core is evaporated before it can be measured and that all of the measured water is surface drainage, the flow of the two small springs and the leakage through the foundation grout curtain.

Since the dam was first closed the maximum settlement has been 2.85 ft. vertical and 1.53 ft. horizontal. The settlement has been a smooth curve from the abutments to the highest section of the dam. There has been no measurable differential settlement. A graph of the settlement vs. time shows that the rate of settlement has decreased so that it will soon be too small to measure. (Fig. 15)

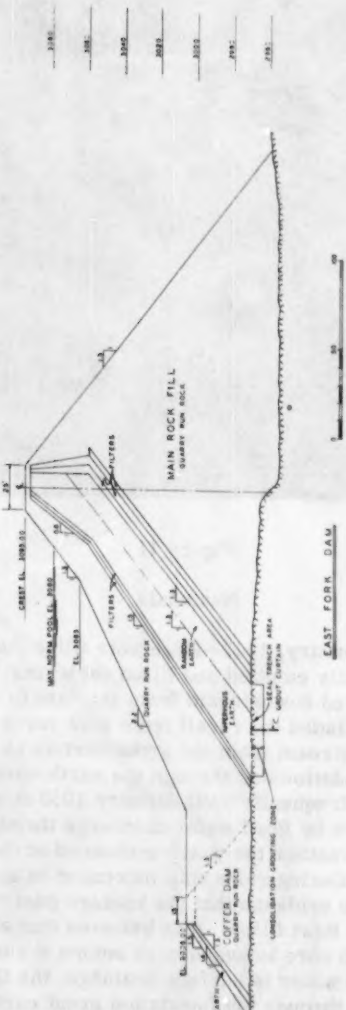

EAST FORK DAM
MAIN ROCK FILL
QUARRY RUN ROCK
COFFER DAM
QUARRY RUN ROCK
FILTERS
FILTERS
IMPERVIOUS EARTH
RANDOM EARTH
SEAL TRENCH AREA
GROUT CURTAIN
CONSOLIDATION GROUTING ZONE
SCALE IN FEET

Figure 12

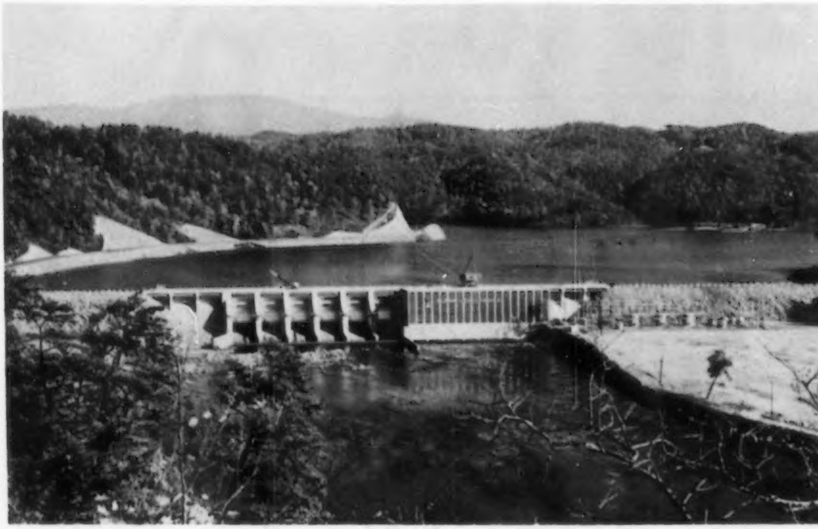


Figure 13

On three separate occasions flood water has passed through the spillway for periods of several days each. Maximum discharge was 3,200 c.f.s. At the lower end of the spillway channel the water flowed down the steep slope to the original stream bed and eroded all the loose material from this slope, as it was expected to do. No erosion occurred in the unlined spillway channel.

Spillway flow is controlled by four Tainter gates and a short section of fuse plug. One of these gates is automatically controlled by a float mechanism.

Opening and closing the spillway gates and operation of the gates controlling the flow of water into the conduit to the turbine are the only operating procedures which have been required at the Nantahala Dam.

In the sixteen years of operation no repairs whatever have been made to the dam. No maintenance, whatever has been required and it is believed that no maintenance except painting the spillway gates will be required in the foreseeable future.

Queen's Creek

Completed in 1948. Reservoir filled in 1948. Queen's Creek Dam was built on the alluvial material filling the creek bed. The seal trench was not excavated to rock and only a limited amount of foundation grouting was done. All the water passing the dam is believed to be leakage through the foundation. It amounts to 23 gallons per minute with a full reservoir. It has not increased since the reservoir was first filled.

Queen's Creek Dam has a rather thick impervious core made of relatively poor material and only a thin rock blanket upstream from the core. The

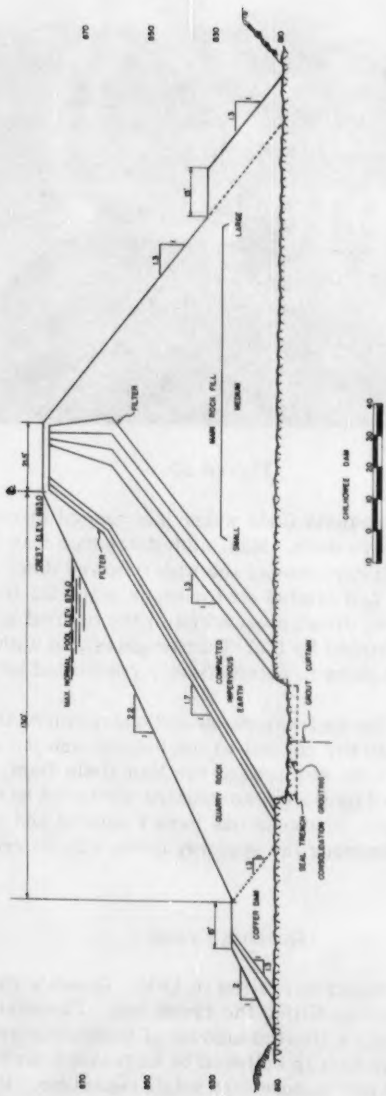

CHILHOWEE DAM
MAIN ROCK FILL
FILTER
FILTER
COMPACTED IMPERVIOUS EARTH
QUARRY ROCK
SEAL TRENCH
CONSOLIDATION GROUTING
GROUT CURTAIN
SMALL
MEDIUM
LARGE

Figure 14

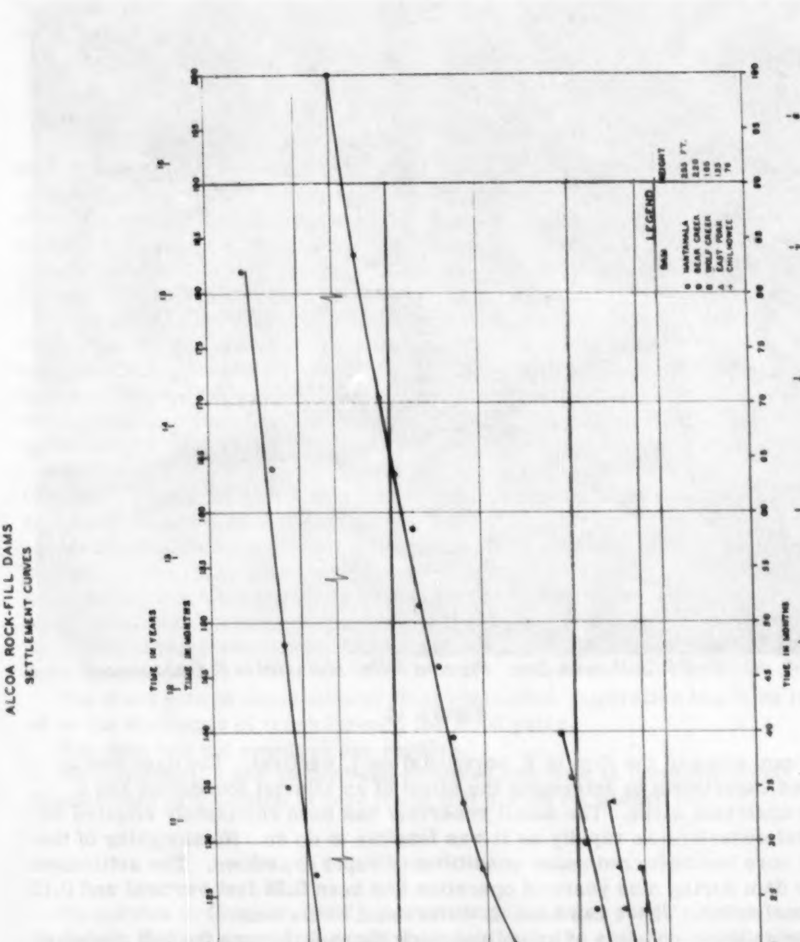
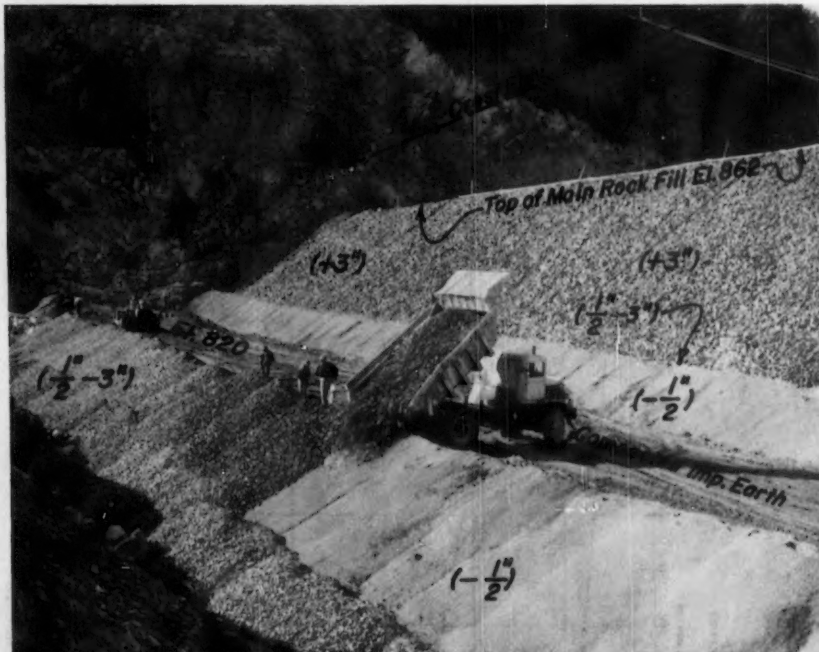

ALCOA ROCK-FILL DAMS
SETTLEMENT CURVES
TIME IN YEARS
TIME IN MONTHS
LEGEND

Figure 15



YD77.4 10-16-56 Chilhowee Dev. Placing Filter Material in S. Embankment

Figure 16

upstream slope of the dam is 2. horizontal on 1. vertical. The dam was in part an experiment to determine the effect of an alluvial foundation and a steep upstream slope. The small reservoir has been completely emptied on several occasions as rapidly as it was feasible to do so. No sloughing of the earth core has occurred under conditions of rapid drawdown. The settlement of the dam during nine years of operation has been 0.28 feet vertical and 0.18 feet horizontal. There has been no differential settlement.

The spillway consists of an unlined rock channel through the left abutment ending in a cascade a short distance below the toe of the dam. This spillway channel has a capacity of 14,000 c.f.s. and it is closed by two fuse plugs which will wash out if overtopped. A conduit under the dam takes water to the pipe line leading to the turbine and also to a valve having a flood discharge capacity of 100 c.f.s. The flood discharge conduit has been used several times. The spillway has not been used.

The only operation required by the Queen's Creek Dam has been to open the flood discharge valve during very heavy rains.

The only repair work necessary on Queen's Creek Dam was performed on the concrete conduit just above the flood discharge valve where a crack developed. This crack was sealed and the conduit is now operating satisfactorily.

No other maintenance has been required and the only maintenance which can be foreseen is replacing the fuse plugs if and when they are washed out by especially large floods.

Cedar Cliff

Completed August, 1952. Reservoir filled shortly thereafter. The foundation and abutments of the Cedar Cliff Dam appeared to be quite tight and only a small amount of grouting in the seal trench was done to reduce construction costs. The leakage measured immediately below the dam was 32 gallons per minute with the reservoir filled and in dry weather. It has not increased since the reservoir was filled. It is believed that all of the measured leakage comes through the foundation.

Soon after Cedar Cliff Dam was completed, a crack developed along the crest between the upstream rock blanket and the impervious core. The reservoir was later completely emptied at a rapid rate and the entire dam examined for sloughing or for areas of distress. No areas of sloughing or distress were found to exist. The crack is believed to be a result of not sluicing the upstream rock blanket and thereby not obtaining rock to rock contact throughout the blanket. The dam performance to date has been completely satisfactory. Settlement during five and one-half years of operation has been 1.08 feet vertical and 0.26 feet horizontal.

Several floods have passed through the gated spillway channel and over the cascade to the river channel close to the downstream toe of the dam. The cascade has operated perfectly to absorb the energy of the falling water with some small enlargement of the area at the bottom of the cascade. No flood in excess of the gate capacity has so far occurred and the fuse plugs have not been washed out.

The flood gate is automatically float controlled. Operation has been limited to the discharge of trash through the flood gates.

The dam has not required any repairs.

No maintenance has so far been required and none is foreseen except the replacement of a section of fuse plugs if it should be washed out.

Bear Creek

Completed in October, 1953. Reservoir filled some three months later. Only a limited amount of grouting was done in the seal trench. Leakage past the dam with the reservoir filled and dry weather has been consistently 60 gallons per minute. It is believed that all the measured water passes through the foundation. The rate of leakage has not changed since the reservoir was filled.

The Bear Creek Dam was placed in one high lift except for the top 40 feet. The main rock fill was heavily sluiced but the upstream rock blanket was not sluiced at all. As at Cedar Cliff a crack developed along the crest between the upstream rock blanket and the impervious earth core. Experience at Cedar Cliff and the performance of Bear Creek Dam have shown that this crack in no way affects the operation of the dam. The maximum settlement has been 0.95 feet vertical and 0.54 feet horizontal. The reservoir has been drawn down on several occasions but has normally operated within a few feet of full.

The Bear Creek spillway is an unlined channel in the rock of the right abutment. It is controlled by one float operated automatic Tainter Gate with a "Hundred Year Flood" capacity and a series of fuse plugs with a spillway design flood capacity. It terminates in a series of three cascades cut in the rock which returns the flood water to the river bed. The spillway has been used several times; once for a considerable period with a maximum discharge of 3,800 c.f.s. There has been no erosion of the spillway channel and no enlargement of the cascades. The fuse plugs have so far remained in place.

The only operation required at this dam is the discharge of trash through the spillway gate.

No repairs have been required.

No maintenance has been required and none is foreseen in the future except painting the spillway gate and replacing the section of fuse plug if it should be washed out.

Wolf Creek Dam

Completed in May, 1955—Reservoir filled shortly thereafter.

The foundation at Wolf Creek is relatively less watertight than the foundation of the dams previously constructed. Considerable grouting was required to limit the loss of water. The leakage measured below the dam with the reservoir full in dry weather is 110 gallons per minute. This relatively large leakage could have been reduced by further grouting in the seal trench. The leakage does not endanger the dam structurally and the cost of the grouting required to substantially reduce it would have been more than the value of the water saved.

Settlement of the Wolf Creek Dam since the reservoir was filled has been 0.44 feet vertical and 0.28 feet horizontal. The settlement varies uniformly from the highest section of the dam to the abutments. There has been no differential settlement.

The Wolf Creek spillway is an unlined channel cut in the rock of the right abutment. The flow in the spillway is controlled by one float operated Tainter Gate and two fuse plugs. The channel ends in two cascades which returns the water to the stream bed not far below the toe of the dam. Several minor floods have passed through the spillway with no erosion and no damage. The fuse plugs have remained intact.

The dam has not required any operation.

No repairs have been required.

No maintenance has been required. The only foreseeable maintenance is painting the Tainter gate and replacing the fuse plug if and when it is washed out.

East Fork

Completed in May, 1955. The reservoir was filled shortly thereafter.

The rock foundation for the East Fork Dam appeared relatively tight and only a moderate amount of grouting was done in the seal trench. The leakage past the East Fork Dam with the reservoir filled in dry weather is 20 gallons per minute all of which is believed to be through the foundation.

The settlement of the East Fork Dam has been 0.43 feet vertical and 0.16 feet horizontal diminishing uniformly from the highest section of the dam to the abutments. There has been no differential settlement.

The East Fork spillway is an unlined channel in the rock of the right abutment. The flow in the spillway is controlled by a float operated automatic Tainter Gate and two fuse plugs. The spillway terminates in a cascade. Several small floods have passed through the spillway without damage. The fuse plugs have remained intact.

No operation has been required at the dam. Some badly cracked rock in the right abutment has been grouted after the dam was placed in operation.

No maintenance has been required and none can be foreseen except painting the Tainter Gate and replacing a section of fuse plug if it should be washed out.

Chilhowee

Completed in August, 1957. Reservoir filled shortly thereafter.

The foundation of the rockfill section of Chilhowee Dam appeared to be quite tight. The seal trench was thoroughly grouted. No leakage has been measured or observed since the dam was placed in operation.

Settlement of the rockfill section of Chilhowee Dam has been 0.16 feet vertical and 0.12 feet horizontal. There has been no differential settlement.

All the flood waters are passed through the concrete spillway section of the dam and do not affect the rockfill section.

The rockfill section requires no operation.

The rockfill section has not required any repairs and no maintenance. No repairs or maintenance can be foreseen in the future.

CONCLUSIONS

All of the rockfill dams leak but only a very small amount. The leakage does no harm to the structure and its economic value is negligible. It is believed that none of the measured leakage passes through the earth core of any of the dams.

All rockfill dams settle. Settlement of Alcoa's rockfill dams has been less than allowed for in the design. The rate of settlement decreases with time so that the settlement will shortly become too small to measure. (Fig. 15)

While the service life of these dams has been short, the spillways have handled all the floods without damage. The spillways are believed to be ample to pass any flood which can reasonably be expected to occur without damage to themselves or to the dam. The use of one or more cascades to return the water from the spillway channel to the stream bed is very effective in limiting the amount of material moved by the flood water.

The dams themselves have required no operation procedures except the handling of trash through the sluice gates. The dams have required no repairs or maintenance nor can any be foreseen in the future except the maintenance required by the sluice gates and the replacements of a fuse plug if it should be washed out.

One other important factor of performance may well be mentioned. The cost of Alcoa's rockfill dams has been much less than that of any other type of structure for these sites. The resulting low annual cost has been very satisfactory.

The sloping core rockfill dams so far designed and constructed by Alcoa have performed to the entire satisfaction of their owners and operators.

The Power Engineering Division of the Aluminum Company of America, with the cooperation of Mr. B. J. Fletcher, Vice President, General Manager of Engineering has furnished the information contained herein.

List of Illustrations

Paper 1745 August, 1958 PO 4

Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: PERFORMANCE OF MUD MOUNTAIN DAM

Allen S. Cary,[1] A.M. ASCE
(Proc. Paper 1745)

FOREWARD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Mud Mountain Dam, White River, Washington, an earth and rock fill structure 400 feet high, has settled in the core zone almost exactly as predicted during design studies which were based on large scale consolidation tests. Differential settlement between core and shell zones caused cracking at the crest parallel to the axis of the dam with rock shells settling about 18 inches more than the core.

INTRODUCTION

Mud Mountain Dam is a flood control dam constructed 1939 to 1941 by the Seattle District Corps of Engineers on the White River, Washington near

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1745 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

1. Chf. Foundation and Materials Branch, U.S. Army Engr. District, Seattle, Seattle, Wash.

Enumclaw, 50 miles southeast of Seattle. White River drains the glaciers on the northeast and north sides of Mount Rainier. It is a swift mountain stream falling 400 feet in the seven mile length of reservoir above the dam.

Site

The dam site is a narrow rock canyon with the right side at the axis rising vertically from elevation 870 to 1090 and the left side sloping about 1/3 on 1 from 870 to 1150. Above the top of rock at elevation 1130, extremely compact glacial till occurred which sloped at about 1 1/2 on 1 to elevation about 1300. The till continues to elevation about 1250 and is overlain by glacial lake silt and clay through which the spillway channel was excavated on the right bank.

Structure

Mud Mountain Dam is a rock fill structure 400 feet high with a central rolled earth core. Slopes are symmetrical up and downstream about the central axis, the top 150 feet of the dam having 1 on 1-3/4 slopes and the lower 250 feet having 1 on 2-1/4 slopes. The central core is 30 feet thick at the top and 150 feet thick at the base. Transition zones on either side of the core are each 10 feet thick at the top and 50 feet thick at the base.

Materials

Materials used in the dam were:

a. Rock fill, an andesite tuffaceous to very hard.

b. Transition, crushed fine grained granite.

c. Core, pit-run sand and gravel with an admixture of 20 percent silty glacial till-like material to reduce permeability. All cobbles over 6 inches were removed from the core material.

Construction

During construction of the dam all riverbed materials were removed to the surface of bedrock under the entire dam, a distance of about 1600 feet along the stream bed. Canyon walls were sloped to insure wedging action as the fill settled and to eliminate overhangs in the rock. The core zone was constructed in 6-inch compacted layers and the transition zones in 12-inch layers. Rock fill was dumped in 40-foot lifts and sluiced with 1-1/2 cubic yards of water per cubic yard of rock.

Settlement Gages

In order to measure the settlement of the dam, gages were installed along the axis of the dam in the core as construction proceeded at 100,200 and 300 feet above the bedrock surface. Monuments were also installed along the crest of the dam as well as at 50-foot intervals on the upstream and downstream slopes of the rock shell zones.

Settlement

Settlement computations based on large scale consolidation tests indicated that the maximum settlement of the core section following completion of the dam would be 2.9 feet. The crest was overbuilt a maximum of 4 feet or 1 percent of the height of fill along the axis. Settlement measurements from the date of completion of the dam in 1941 continuing until 1950 indicated a total maximum settlement of 2.7 feet at the crest of the dam over the deepest canyon section, 400 feet. Thus it can be seen that actual settlement was very close to the theoretical in 1950. No further measurements have been taken. The monuments which were placed at 50-foot intervals on the rock fill slopes were checked for horizontal movement until 1950. Horizontal movement varied from zero to a maximum of 0.90 foot. No advance predictions had been made on this movement.

Cracks Along Top of the Dam Between Core and Shell

In August 1942 cracks appeared on the crest of the dam along the junction between the core and the upstream and downstream transition zones parallel to the axis. Test pits revealed that the cracks were completely indiscernible at 6 feet depth. The cracks were excavated and backfilled but continued movement through 1948 reopened the cracks to a maximum of about 4 inches wide. The shell zones had settled a total of about 18 inches below the core zones. The differential settlement between core and rock shell zones was not considered serious and since 1948 there has been no measurable differential movement.

Discussion and Conclusion

The maximum pool level which has been reached in flood control operation has been 1117 feet — 98 feet below spillway crest of 1215 feet. No study of the effect of full pool on behavior of the dam has been made and none seems warranted in view of the close parallel between predicted and actual settlement. It is probably that the settlement of the rock shells could have been reduced if more sluicing water had been used and also if the rock had all been very hard such as basalt instead of some tuffaceous andesite.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: WISHON AND COURTRIGHT CONCRETE FACE DAMS

J. Barry Cooke,[1] M. ASCE
(Proc. Paper 1746)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

Design considerations and construction of the two approximately 300-foot high dams are discussed. All changes in design and construction from previous P. G. and E. dams have been made to lower the cost without affecting the safety of the dams. Wishon was filled in May 1958 and initial crest settlement data is presented.

INTRODUCTION

Wishon and Courtright dams are located on the North Fork watershed of the Kings River about 60 miles from Fresno, California. Together they will

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1746 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

1. Senior Engr., Pacific Gas and Electric Co., San Francisco, Calif.

store 251,300 acre feet of water which will fall through 5645 feet of head in Pacific Gas and Electric Company's Haas, Balch and Kings River hydroelectric plants.(2,8) Physical data in comparison with the Company's Salt Springs and Lower Bear River dams is presented in Table I, and discussion of the two earlier dams is presented in reference (10).

Wishon Dam was completed on May 10, 1958, and the reservoir completely filled on May 27, 1958. Courtright will be completed in 1958. The two dams were designed concurrently and are therefore similar. Wishon will be reviewed in detail and Courtright will be discussed only as it differs from Wishon.

Wishon Design

The selection of the concrete face rockfill type of dam was made on the basis of its being the lowest cost dam considering the site, availability of materials, and reservoir operating conditions. The site is exposed glaciated granite with essentially no overburden or decomposed granite, Fig. 7. Exploration for core materials, for core rockfill types of dam, failed to locate satisfactory materials in meadows or in zones of decomposed granite in a radius of about 20 miles. Reservoir operation will normally unwater the top two-thirds of the face each year and all may be unwatered, if necessary, without impairing operation.

The cost of a concrete gravity dam was estimated at twice that of a concrete face rockfill. A thin central core rockfill and a sloping core rockfill were estimated to cost about the same as a concrete face rockfill. A decision between types of rockfill based on reasons other than economic was not required since core material was not available.

Having selected the concrete face rockfill type of dam, the experience record of Salt Springs, and of the Lower Bear River dams, was thoroughly reviewed. The review covered construction as well as design and performance, with the objective of specifying a dam that could be constructed by modern methods and in a straightforward manner. All changes from Lower Bear River design were made to obtain a more economical design and to simplify construction. The changes do not alter the safety of the dam; they should not adversely affect the performance; and they substantially lowered the cost. The main changes in order of the amount of saving are:

(1) Thinner placed rock.
(2) Steeper upstream face.
(3) Less rigid placed rock specification.
(4) Use of face lifts for placed rock and face flab construction access.
(5) Elimination of keyways in the placed rock.
(6) Permit placed rock points to project to the plane of the reinforcing steel.
(7) Simplified joints.

Diversion, Outlet Works and Spillway

Diversion Tunnel.—The size and alignment of diversion tunnel was given to the bidders. In the first runoff season, the partially completed dam, with broad crest width and no concrete face, was to serve as cofferdam. In the second runoff season, the permanent outlet works was to be installed in order to act as a spillway, to permit storage of water with the partially completed dam. The design capacities were such that several floods of record would have caused not more than 2000 cfs to pass through the partially completed rockfill. It was considered that such would not endanger this dam of large rock, and the economy in size of tunnel and outlet works was obtained. Circumstances were such that actually no flows passed through the rockfill.

Outlet Works.—The capacity was governed by the use of the outlet works as a spillway between the second and third construction season in order to obtain early storage, as discussed above. An 84-inch Howell-Bunger valve and a 120-inch butterfly valve are installed in a chamber in the diversion tunnel, as illustrated and described in reference (8). The diversion tunnel leading to and from the valve chamber is left unlined.

Spillway.—The spillway, Fig. 1, consists of six 40-foot by 11-foot radial gates that discharge into the spillway quarry, an unlined rock channel. The channel discharges onto exposed bedrock and the flow reaches the river a quarter of a mile downstream from the dam. The design flow is 30,000 cfs for the maximum storage and flood level of 6550, and freeboard is provided by the 4-foot coping wall, Fig. 5. The concrete face rockfill dam is not considered to require as much freeboard as an earth or earth core rockfill dam. The tops of five auxiliary concrete dams are only one foot above maximum storage level, and the spillway channel (quarry floor) is sloped to take increased flow without causing backwater effect at the spillway gate structure. These two features give excess spillway capacity at no cost.

Auxiliary Dams

Five concrete gravity auxiliary dams are required along a glaciated granite ridge that forms the right abutment of the main dam. The maximum height is 30 feet and total crest length 240 feet. The crest being 1 foot above maximum storage level and 3 feet below top of coping wall, the auxiliary dams are of minimum cost and provide excess spillway capacity.

Cross Section and Alignment

The cross section of the dam, Fig. 1, is similar to Salt Springs, Table 1, but with slightly steeper upstream face. At Lower Bear River dams the construction experience showed that the 1:1 face of Dam No. 2 could be constructed at essentially the same unit costs as the 1.3:1 face of Dam No. 1. It was considered that the use of "face lifts" proposed for Wishon would make the 1:1 face more easily constructed, and that the reduced quantities would directly represent reduced cost.

Upstream Face.—The upstream face slope, Fig. 1, may be considered to be 1:1 in the upper portion and 1.3:1 in the lower portion of the dam. At elev. 6400, 150 feet from the crest, a transition between the two slopes is made by a 40-foot height of slab at 1.1:1 above elev. 6400 and another at 1.2:1 below. The 1:1 slope is stable with a minimum thickness of placed rock; can be constructed conveniently; and concrete can be placed rapidly by slip forms. The

TABLE I

STATISTICAL DATA FOR FIVE
P. G. AND E. CO. ROCKFILL DAMS

	UNIT	SALT SPRINGS	LOWER BEAR RIVER NO. 1	LOWER BEAR RIVER NO. 2	WISHON MAIN	WISHON WING	COURT-RIGHT
Year Completed	YR	1931	1952	1952	1958	1958	1959
Storage	AF	142,000	49,000		128,500		123,300
Crest Elevation	FT	3958	5820		6500		8188
Height (Above Foundation)	FT	332	250	155	296	166	317
Crest Length	FT	1300	960	865	1415	1915	900
Camber (Max.)	FT	6	3.3	1.1	4.0	2.2	4.2
Surface Area of Face	SF	380,000	190,000	90,000	650,000		220,000
Crest Width	FT	15	20	20	20	20	16
Max. Height (To Design Crest)	FT						
Streambed at Axis		302	234	107	256	151	295
Foundation at Axis		328	234	150	260	158	295
Streambed		313	250	125	256	160	317
Foundation		332	250	155	296	166	317
Cutoff Line		318	221	137	250	130	289
Cutoff Trench		348	238	148	288	152	337
Slopes (Horiz. to 1 Vert.)							
Upstream		1.4 to 1.1	1.3	1.0	1.3 to 1.0	1.0	1.3 to 1.0
*Downstream		1.4	Nat. to 1.4	Nat.	Nat. to 1.4	Nat.	Nat. to 1.4
Placed Rock Thickness	FT						
Top		15	10	10	7.8	7.8	7.8
Down 100 Ft		15	15	15	7.8	7.8	7.8
Down 150 Ft		15	17.5	-	10	-	10
Down 200 Ft		15	20	-	10	-	10
Max. (Cutoff Line)		15	21	16.9	11.6	7.8	11.6
Quantities (1000 CY)							
Excavation		312	10	138	270		60
Rock-Dumped		2,744	900	349	3,530		1,540
Placed		224	89	43	200		77
Total		2,968	989	392	3,730		1,617
Placed:Dumped	%	8.1	9.9	12.0	5.7		5.0
**Concrete		45	23	11	86		38

* Nat. = Natural = 1.3 Plus

** Includes cutoff, coping, face slab theoretical and excess thickness, and abutment gravity sections.

1.3:1 slope was preferred below a height of 150 feet to transmit the higher water pressures more directly to the foundation through a shorter distance in the rockfill; to better transmit the horizontal component of water pressure to the foundation in the vicinity of the cutoff wall; and to show increased respect for the higher dam. Adjacent to the cutoff, the rocks underlying the slab must transmit the horizontal component of water pressure to bedrock with minimum yielding or movement. With the 1.3:1 face slope, this is accomplished with a ratio of vertical to horizontal force of 1.3:1, which is more favorable than 1:1. The 1.3:1 slope reduces face movements in the heavily loaded lower portion of the dam, by transmitting the water load to the foundation through a shorter distance in dumped rock. The cost of the flatter slope is not great since it occurs in the narrow portion of the site. The full height of the wing section of maximum 166-foot height, Fig. 1 and 6, is at 1:1 upstream slope. The terms "wing section" and "main section", Fig. 1, are used to identify the two rather independent portions of Wishon Dam.

Downstream Slope.—The downstream slope is essentially the natural dumped slope. Because the natural slope is irregular and unknown, the first lifts are carried out to a toe located by a 1.4:1 slope from a 12-foot wide theoretical crest at the storage level of 6550. The upper portion of the slope is the natural slope that results from constructing the 20-foot crest width to the constructed crest elevation, Fig. 1 and 5. The 20-foot crest width provides for two-way traffic and otherwise is wider than considered necessary for the concrete face rockfill dam.

Alignment.—The horizontal alignment of the dam is based largely on determining minimum quantities and cost. For the concrete face dam, a location giving minimum face area, rather than minimum volume, tends to govern. It is noted, Fig. 1, that the cutoff is along the ridge on both abutments, which requires more dumped fill, but gives a dam having minimum face area and minimum cost. The dam is on a 5000-foot radius curve. The face joints of the main and wing sections are normal to two chords that intersect at Station 15 + 00, Fig. 2. The curvature is somewhat more than required for settlement and appearance in order to obtain minimum cost, by adapting the long dam to the site contours. The vertical alignment provides a crown four feet above design crest at the 296-foot high maximum section.

Cutoff

Abutment Gravity Sections.—At each end of the dam a concrete gravity section, Fig. 2, is used up to a height of 30 feet, at about which height the cost of a gravity section equals that of the concrete face rockfill. The upstream face is 1:1 and, with copper joint, is connected to the first face slab of the rockfill dam. The use of this gravity section provides economical and straightforward ends for the dam. The coping wall continues, dowelled to bedrock, until it is of zero height.

Concrete Face Cutoff.—The cutoff is shown on Fig. 3 in comparison with the cutoff of Salt Springs and Lower Bear River dams. It is discouraging to excavate sound rock and replace it with concrete, and the trend has been to reduce the depth of cutoff in sound bedrock. In seamy rock the cutoff is carried to greater depths. Even with careful blasting, there is a tendency to loosen adjacent rock and the cost of the excavation is very high. The trench is upstream from the cutoff line to eliminate uplift. If inferior rock were encountered in local areas it was intended to provide an upstream slab or to

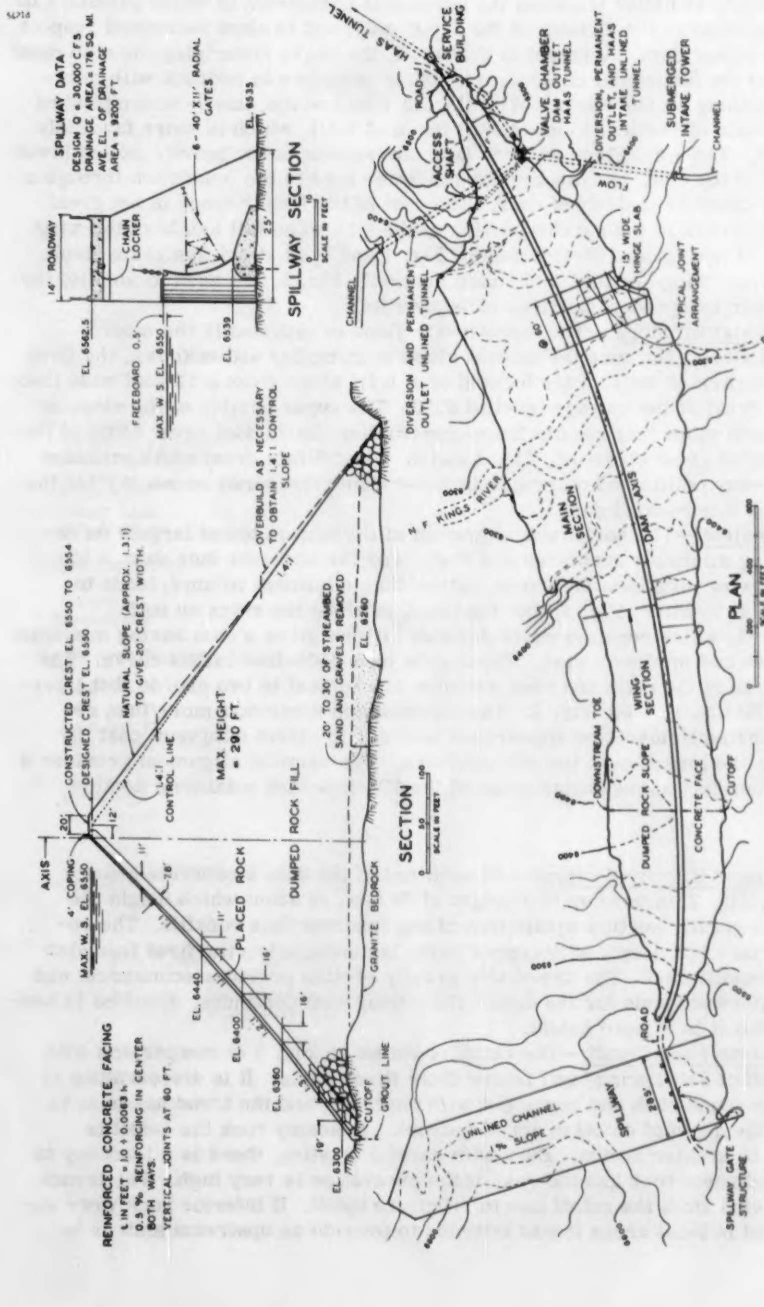


FIG. 1 WISHON DAM PLAN, SECTION AND SPILLWAY SECTION

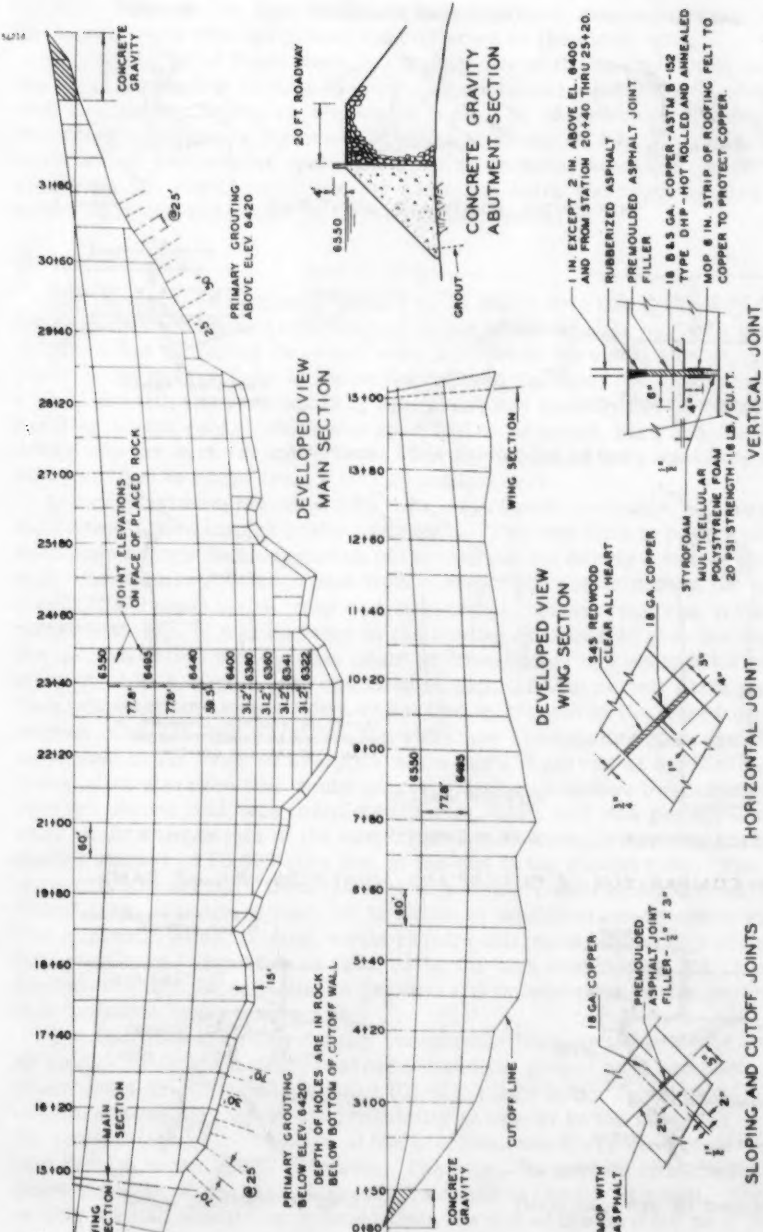


FIG. 2 JOINT ARRANGEMENT AND DETAILS - GROUTING PATTERN - ABUTMENT GRAVITY SECTION - WISHON DAM

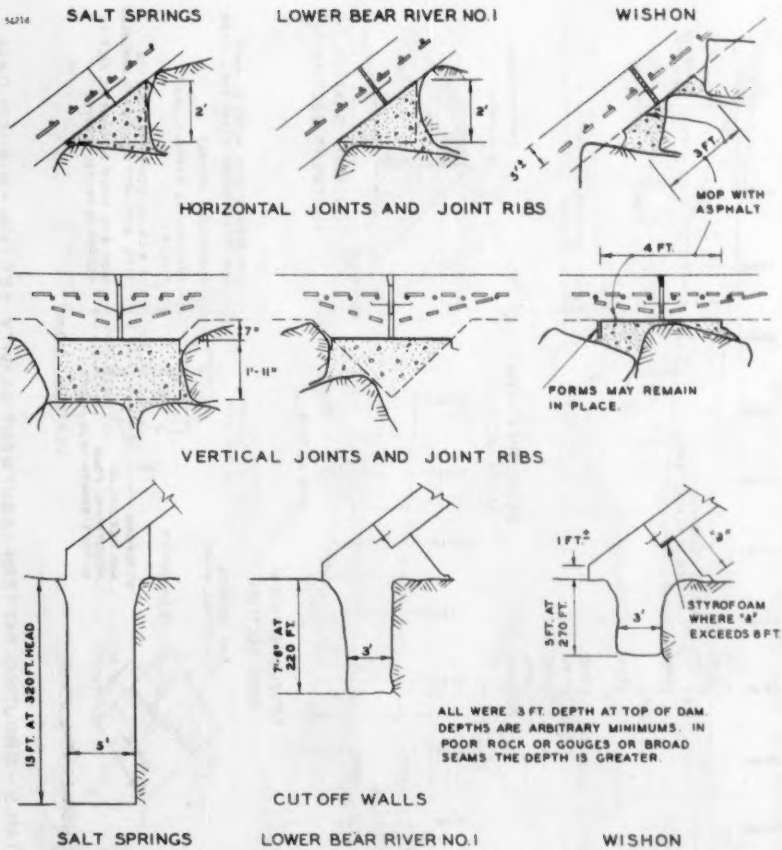


FIG. 3 COMPARISON OF CUTOFF AND JOINT RIBS - P.G.AND E. DAMS

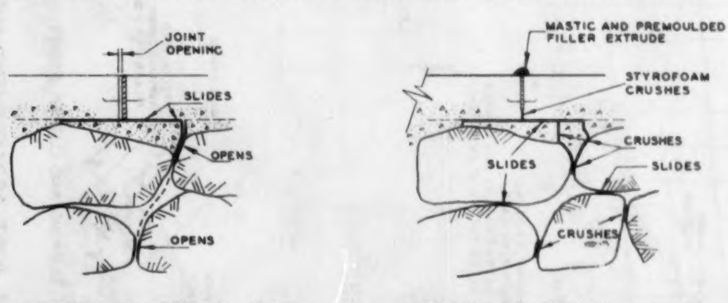


FIG. 4 ILLUSTRATION OF FACE JOINT MOVEMENT - WISHON DAM

bed the lower layer of placed rock in concrete to obtain a longer percolation path, but this was not required. Across some seamy or gouged zones, the cutoff wall became as high as 15 feet and styrofoam was used, Fig. 3,to allow the face to yield with the placed rock adjacent to the cutoff wall.

Grouting.—Grout pipes were arbitrarily placed in the cutoff wall normal to the cutoff line and at 25-foot spacing, and additional pipes located so that a drill hole will intersect visible seams some 5 to 10 feet below the cutoff trench. The arbitrary pattern of grouting is shown on Fig. 2. The deeper holes at 100-foot spacing were cored and were water pressure tested before grouting. The requirement was for additional holes and regrouting as determined by the grout take from the primary holes.

Dumped Rockfill

Specification.—Selection of quarry sites was left to the contractor, except for specific excavation requirements in the spillway area, and with the restriction that no quarry be closer than 200 feet to the cutoff line or outlet tunnel if the quarry floor is below the crest of the dam. To assure large size rock in the fill, the rock handling equipment was specified to be capable of handling 15-ton rocks. Rock was specified to be sound, hard and durable with definitions for each term. At least 50% (by weight) of the rock in the dam was specified to range from 0.5 to 10 ton and over.

Lifts.—Maximum height of lifts are considered desirable both to the performance of the dam and to the contractor. The high lifts are associated with maximum energy and compaction going into the fill during construction. The high lifts require minimum haul road construction and minimum lift surface area to be cleaned up. A proposed, acceptable, but not required, lift arrangement, Fig. 5, was included in the bidding drawings to give the contractors an idea of one way the dam could be constructed in a straightforward manner. Lift 1, Fig. 5, was specified at elev. 6380 to permit early placed rock construction and provide a cofferdam by the end of the first construction season. The 140-foot minimum top width was to minimize upstream face movement in the event of overpour while this lift served as a cofferdam. Lift 2 was at an elevation that would assure completion across the canyon by the time the placed rock was completed to elev. 6380, and thus permit continuous work on all components of the dam. Lift 3 is at a required elevation to assure partial storage of 80,000 acre feet at the end of the second year. The three main lifts are set back from the face to permit "construction access" or "face" lifts. The top of each lift is shown at minimum construction widths. The minimum width, if used, would require minimum lift surface cleanup and give maximum compaction to the dam by the high overlapping fill. However, the benefits are not essential to the dam and broader widths are permitted if they facilitate construction.

The face lifts are theoretically undesirable because the low lifts do not get as much settlement during construction and the placed rock and concrete are constructed on the freshly dumped fill of the face lifts. However, since these considerations only affect the probability of cracks in the face, and not safety, the method is adopted because of the practical and lower cost construction that results in the use of face lifts. The face lifts may be constructed rapidly since the area of the "parallelogram" section of the lift is small. Since there is only a small amount of dumping from the end of the face lift as it progresses, not many fines accumulate and lift surface cleanup is not a problem. The

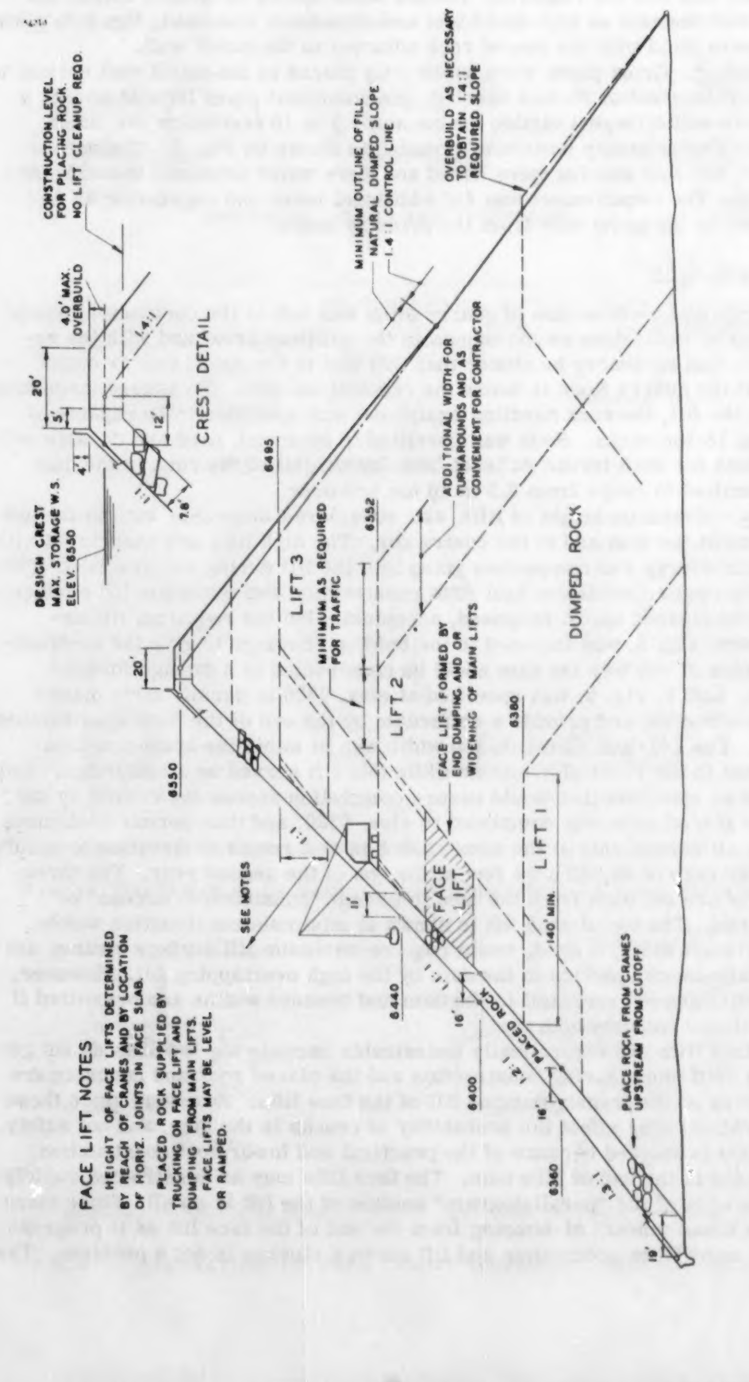


FIG. 5 PROPOSED CONSTRUCTION METHOD - MAIN SECTION - WISHON DAM

height and location of face lifts is as governed by the location of concrete joints and by the reach of the cranes for placing rock.

Sluicing.—Sluicing water was specified to be applied at the times and places where rock is being dumped in a volume three times as great as the volume of rock being dumped. Sluicing is also required, as may be directed, on visible surface zones of fines that may develop. A pressure of 100 psi at the nozzles of 2 to 3-inch diameter was required, which produced jets of 2.5 to 5.5 cfs. A pressure gage was required at each monitor. The action of the powerful jets in excavating pockets of spalls and distributing the spalls into voids of the large rocks is considered as desirable a feature of sluicing as the washing of small fines into the rockfill.

Placed Rock

The placed rock section at Wishon is somewhat thinner than used on previous high rockfill dams of the P. G. and E. Company. It is considered that the thickness does not involve stability for the slopes of 1:1 or flatter than 1:1; and that, with the quality of the well sluiced dumped fill, the thickness should have little effect upon the probability of cracks in the face. The 11-foot horizontal thickness (7.8-foot normal thickness) in the upper portion of the dam, Fig. 1, was adopted as a practicable construction minimum to catch the dumped rocks with the steeper than natural upstream surface slope. However, in the lower portion of the dam, the placed rock was increased to about 11-foot normal thickness, in spite of the flatter face slopes. This is still considerably thinner than at Salt Springs and Lower Bear River, but is thicker than the 7.8-foot minimum adopted for the upper portion of the dam out of respect for the high water pressures in the lower portion of a 300-foot dam, Table I. Since only a small per cent of the surface area of the dam is in the lower portion of the dam, the increased thicknesses at the bottom of Wishon require only 9.5% more placed rock than if all the placed rock were of 11-foot horizontal thickness. It was decided not to depart too much from previous practice at this time. Since the thickness and specification for placed rock is considered to involve economics and practical construction, and not safety, the construction and performance experience at Wishon may lead to further economies in the future.

Specification.—The drawings, Fig. 1 and 6, specify horizontal thicknesses, since horizontal thicknesses can best be used to lay out and inspect the construction. For the wing section, Fig. 6, the 11-foot thickness is used for the full height of dam. The rock was specified to be selected rock, 1 to 10 ton or more in weight, with average weight of not less than 3 tons. Rocks were required to touch each other in up and downstream direction and be as close as possible laterally, with nearest points of adjacent rock not greater than 6 inches apart. Accessible voids between up and downstream rocks were required to be chinked with large rock, and lateral voids to be filled with rocks that would pass a 2-inch grizzly. Rock points or edges were permitted to project to the plane of the reinforcing steel to minimize the amount of excess concrete beyond the theoretical thickness.

In previous specifications for placed rock, the requirement of lateral as well as up and downstream contact of the placed rocks greatly slowed down production. In writing the Wishon Specifications, it was considered that the possible benefits were not worth the cost, and lateral contact was not

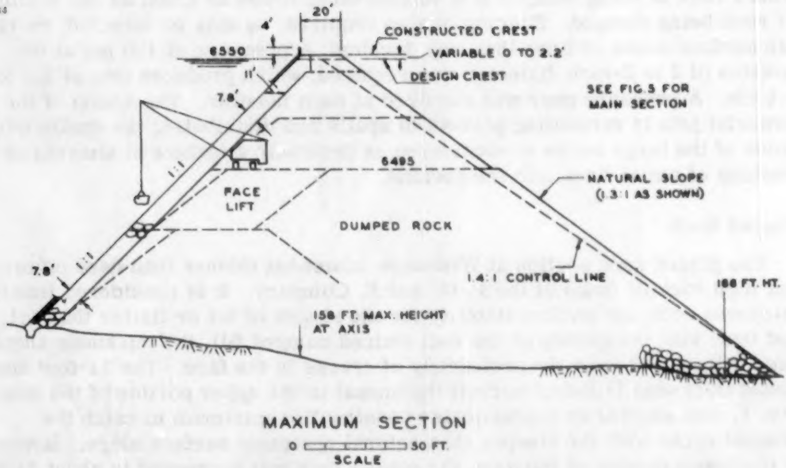


FIG.6 PROPOSED CONSTRUCTION – WING SECTION – WISHON DAM

Fig. 7. Kings River Project Wishon Dam. Dumping rock near the center of the dam. The construction involves hauling 3,687,000 cubic yards of dumped and placed rock in 17 cubic yard trucks. The double monitor is operated by one man.

required. This change involved definite capital cost versus "possible" maintenance cost, and not safety. Vertical contacts are important and occur automatically. Up and downstream contacts transmit the high water pressure into the fill with minimum possibility of local movements.

Concrete Face

The concrete face, at elevation 6550 in the Sierras, will be subject to freezing conditions. The strength is specified on the basis of obtaining a dense frost action resistant concrete and not to resist computed stresses. Concrete was specified to have: 3000 psi 28-day compressive strength; 2-1/2 inch maximum size aggregate; 5-1/2 to 6% air entrainment by volume; and contain a water reducing agent. Cold weather pouring requirements were: not less than 40° F placing temperature in moderate weather; and not less than 50° F temperature for 72 hours after placing in freezing weather.

Thickness and Reinforcing.—The thickness of concrete and the reinforcing, Fig. 1, are the same as at Salt Springs and Lower Bear River. It is arbitrary and has been satisfactory at those dams. One economy in thickness, made at Wishon, was to permit placed rock points and edges to project into the theoretical thickness of slab to the plane of the reinforcing steel. This reduces the amount of concrete beyond the theoretical thickness that is required to fill the surface irregularities of the placed rock.

Joint Arrangement.—The joint arrangement, Fig. 2, is simplified over that used at Lower Bear River to improve performance and at the same time use a minimum total length of joints. The 60-foot vertical joint spacing is used as a practicable spacing for use of a slip form and to give reasonable alignment and flexibility at the cutoff. The wing section, of 166-foot maximum height has only one horizontal joint that divides the face into two pours. In the main section the top two slabs are of 77.8-foot lengths, selected as practical lengths for maximum single pours. The slabs below elevation 6400 are 31.2 feet long because in the lower half of the face the settlement contours are closer together; there is a greater tendency to develop vertical stresses in the face; joint movements are high; and the slabs are thicker and more rigid. The joint that is parallel to, and 15 feet from, the cutoff joint is called a sloping joint. It forms a "hinge slab". The sloping joint is used only where the dam exceeds a 150-foot height since above that point the abutment slopes are gradual at the Wishon site and the movements should be small. In the lower portion, additional vertical joints are used at abrupt changes in cutoff alignment. In general, the lower portion of the dam is given more careful joint treatment since cracks can produce more leakage under the high head and the lower face area will seldom be unwatered.

Joint Ribs.—The term "joint rib" refers to the concrete poured in a strip along face slab joint lines in advance of pouring the face slab. The design adopted for Wishon requires no keyway in the rock and is shown in comparison with Salt Springs and Lower Bear River Dams on Fig. 3. At Wishon there are 20,000 feet of joint, excluding the cutoff joint, and the adopted Wishon design was estimated to cost some $500,000 less than the rib designs previously used on this type of dam. The design was adopted to lower the cost, to speed up the placing of rock, and to improve the functioning of the joints. Keyways have slowed up the placed rock work on previous dams and, due to the irregular sized rock, have required excessive concrete.

The joint rib is essentially a smooth surface of concrete painted with asphalt and located under the joint. It is useful both in construction of the

dam and in the performance of the joint. The rib gives a surface on which the joint can be conveniently, practically, and well constructed, and on which the forms may be supported. The trim contact of joint rib concrete and the form insures a good concrete job beneath the copper seal. Though there are construction benefits from the rib the primary reason for the rib is its function in performance of the joints. The rib makes the opening and closing of a joint possible without damage to the edges of the slabs by separating the edges of the slab from the placed rock. It automatically provides a secondary waterstop in the vicinity of the joint, but this is not a reason for the joint rib.

Fig. 4 illustrates the functioning of a joint. In the opening of a joint, the placed rock must separate under or adjacent to the joint rib. It can occur anywhere under the rib since the slab can slide on the smooth asphalt surface of the rib. It will probably occur as an irregular separation and the rib concrete would probably crack over the underlying separations in the placed rock. In the closing of a joint, some crushing must occur in the rib or in the immediately adjacent placed rock. Some of the crushed material must find a void in the placed rock, compress pieces of the wood form that are left in place, or compress the styrofoam. The rib is of 1500 psi concrete so that it will crush more readily than the face slab. Below the rib some rock points must crush. If the contact is particularly good between rocks immediately under the joint, some sliding of rocks and crushing of points away from the joint can accommodate the movement.

It is apparent that a substantial force is necessary to close a joint, but experience shows that the joint does close before the slab crushes. That the joints resist closing is probably very desirable since it limits movements. At Lower Bear River Dam No. 2 there has been zero joint opening or closing, which indicates that settlement and temperature forces on the face of this small dam have not been great enough to cause movement.[10] On the higher No. 1 dam, nearly all joints have moved.

Joint Details.—There are three types of joints; sloping and cutoff, horizontal, and vertical. Sloping joints are those parallel to and 15 feet from the cutoff joint, and vertical joints are those that go up the face in a vertical plane perpendicular to the axis. Fig. 2 shows the details of the joints and Fig. 5 shows the setting of the horizontal and vertical joints on the joint ribs. The copper is joined by one-half inch laps that are brazed by a thin flowing, low temperature welding alloy rod, flux being applied on the inner surfaces of the lap. This "sweated" joint is more rapidly made and more easily made watertight than a high temperature brazed job.

The cutoff and sloping joints are cold joints with no ability to close. Near the cutoff line there is so little rock just below the slab that it cannot move very much. It is considered that these joints should relieve moments in the slab but otherwise resist movement of the slabs and support the upper slabs. The redwood is used in horizontal joints as a material that will resist the closing of the joint and not extrude. At Lower Bear River[10] all joints except the upper one closed moderately. It is probable that the redwood is not necessary in the elev. 6495 joint, but it is nominal in cost and all horizontal joints were made the same. The use of the Z waterstop in the horizontal joints is an economy over the use of a U waterstop as on previous dams.

The vertical joint is designed to open or close and to permit the slabs to move parallel to the joint with respect to each other. The deep 4-inch U permits larger opening than will occur and permits the copper to wrinkle rather than tear due to differential movements parallel to the joint. The styrofoam

beneath the copper permits complete closing since it compresses to essentially zero volume. The asphalt above the copper can extrude to the surface. Above elev. 6400 the vertical joints of the 8 central vertical joints of the main section were constructed 2 inch open and all other vertical joints one inch open. It is probable that a more simplified vertical joint would be suitable for the wing section based on the performance of the Lower Bear River No. 2 dam(10) but because of the substantial changes in placed rock, the vertical joint detail was not simplified for the wing section at Wishon.

Wishon Construction

The Wishon dam, located at elevation 6550, is snowed in during the winter. A seven month normal construction season may be lengthened to about nine months by fighting snow at both ends of the construction season. In August of 1955 the job was awarded to a joint venture, Morrison-Walsh-Perini, sponsored by Morrison-Knudsen Company. In the remaining several months of the 1955 season, construction started on roads, clearing, opening of quarry and driving of diversion tunnel. Construction in 1956 was primarily streambed excavation, dumping of rock and cut off excavation. The major construction year was 1957, at the end of which Morrison-Knudsen was ahead of schedule, and the remaining work consisted of 40 of the top concrete slabs. It was arranged to complete the dam in winter weather of April-May of 1958 in order to completely fill the 128,000 acre foot reservoir with the 1958 snowmelt runoff. Construction took 21 construction months for the dam of 3350 foot crest, 290 foot maximum height and 3,700,000 cubic yards of rock.

The construction of both Wishon and Courtright is well discussed and illustrated, particularly in regard to methods and equipment, in the construction magazine references (1) (3-7) (9). In this paper construction will be reviewed and illustrated to supplement the more detailed coverage of the references.

Fig. 7 shows the massive granite formation at the site and an interesting stage of construction. The photo is taken from the end of the wing section dumped fill, some of which is in the left foreground. Dumping is proceeding at the 6450 lift, which was carried to the full width of dam rather than partial width as shown on Fig. 5. Placed rock work is proceeding from the face lifts, which were used essentially as proposed on Fig. 5. In the right foreground a portion of the face lift has been scarified to receive the next lift of dumped rock. The pile of fines was formed by monitor and bulldozer in the scarifying operation, and was later removed. From the main lifts the scarified fines are cast over the downstream face. Scarifying is by a bulldozer carrying a monitor that operates at 200 psi. This high pressure is possible by operating at off-shift times when other monitors are shut down. Large rocks may be seen high up on the dumped slopes, which is desirable. It takes well interlocked surface rocks to stop the large rock from sliding further down the slope.

Fig. 8 shows sluicing from cantilevered monitors, and also typical dumped fill. The monitors are located such that they can excavate surface zones of fines as dumping proceeds. Large rocks sliding down the slope cause thin surface slides or movement of rocks, but such surface movements do not endanger the truck rear wheels which are at the edge of the fill. The sluicing operation requires continual effort by inspectors and monitor operators to get the maximum benefit out of sluicing. Inspection of the rockfill shows that the

FIG. 8 TYPICAL ROCKFILL - SLUICING - WISHON DAM

FIG. 9 FACE CONSTRUCTION - WISHON DAM

rocks come to rest generally in secure-wedged contacts rather than point contacts. The sliding or rolling of the heaviest rocks appears to give material compaction effort to the fill.

Fig. 9 shows the surface placed rock joint ribs and some completed slabs. The placed rocks range in weight from 6 to 15 tons and in maximum dimension, 6 to 12 feet. The surface of the placed rock is quite irregular since the cost of excess concrete in the irregularities is less than the cost of obtaining a smoother face surface with irregular sized and shaped rocks. The small rocks are to fill the surface voids to prevent loss of concrete or concrete mortar in the placed rock. The joint ribs are screeded on the surface, not formed. Concrete may run out under the forms, the only requirement being a surface of 4 foot width for vertical joints and 3 foot for horizontal joints, Fig. 3. The form seen at the edge of the completed slab is reusable and is heavily constructed to carry the wheels of the slip form.

Figs. 10 and 11 show the progress during the 9 months of construction in 1957. These isometric drawings have been redrawn from construction planning and progress drawings made by Morrison-Walsh-Perini. It is seen that construction of the three major elements of the dam, dumped rock, placed rock and concrete face, all proceed simultaneously and continuously. The first isometric shows the work accomplished in six months of 1956, which included primarily the streambed excavation, cutoff excavation, and the dumped rock as shown.

In November of 1957 a small crew was maintained during the cold weather to complete all the dumped and placed rock. The required scheduled completion was only to elevation 6495. Figs. 12 and 13 show two general views of construction stages.

Fig. 14 shows the dam nearing completion in April, 1958. In the middle of March, 1958 an Agreement was signed guaranteeing completion of the dam by June 1, 1958, just in time to catch the snow melt runoff. The dam was snowed in, and as snow was removed and work began a series of storms in late March and early April impaired progress. In spite of the discouraging start the dam was finished on May 9, three weeks ahead of the accelerated schedule. The slip form was designed by Morrison-Knudsen Company. It was operated by electric winches on the form and pulled to blocks anchored to large rocks near the top of the pour.

Foundation Excavation.—The streambed material was sand and gravel with cobbles of 4 to 6 inch size, but there were very few boulders. It was decided to remove them because of their small size and they were removed by dragline working under water. Excavation stopped when the dragline began hitting high spots of bedrock. The holes were not unwatered or completely cleaned out. The estimate, based on several drill holes at and upstream from the axis, was 75,000 cu. yds., but 170,000 cu. yds. were removed and most downstream from the axis. Had this been known more consideration would have been given to leaving most of the material downstream from the axis in place with the use of filters.

Cutoff.—It is considered desirable to make the cutoff line straight between vertical joints in order that the slab may be considered to "hinge" at the cutoff joint. High points of the irregular shaped abutment rock were removed down to the cutoff line, and then the 3 foot wide trench was carefully excavated. The cutoff requires drill holes at about 1 foot centers in both directions, light blasting, and hand mucking. The grouting was carried out to the arbitrary pattern, Fig. 2, with additional holes where seams or jointing planes

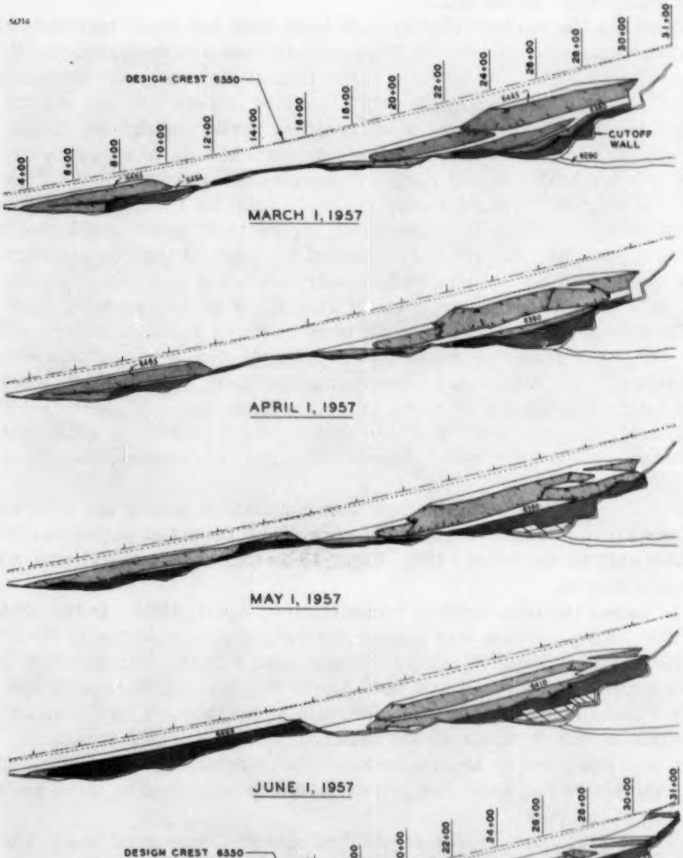


FIG. 10 MONTHLY CONSTRUCTION PROGRESS ISOMETRICS–WISHON DAM

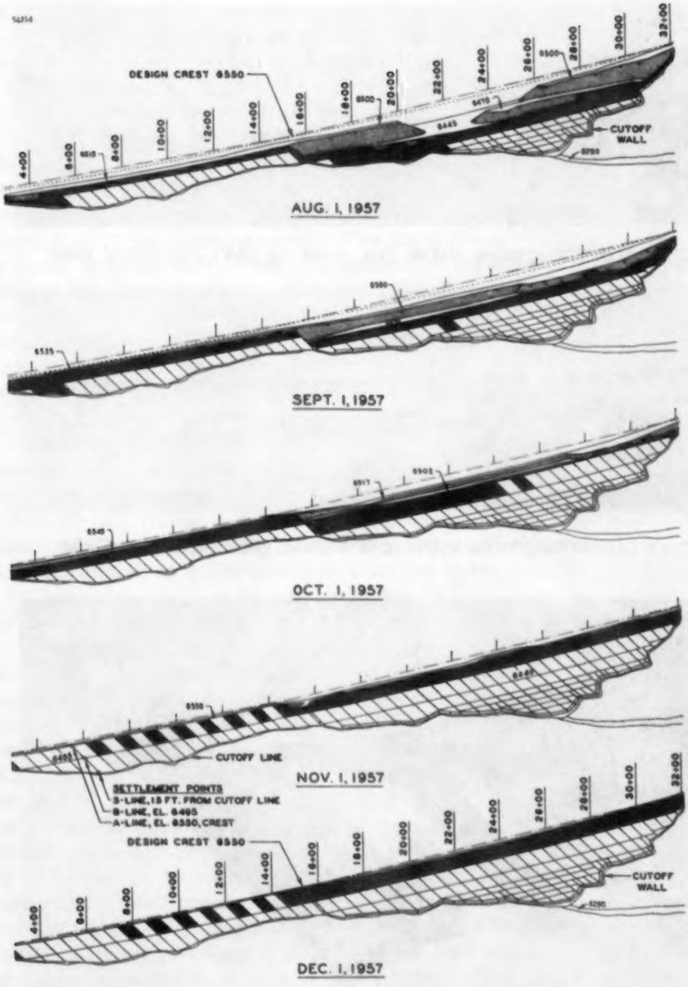


FIG. 11 MONTHLY CONSTRUCTION PROGRESS ISOMETRICS–WISHON DAM

FIG. 12 CONSTRUCTION VIEW ON JUNE 10, 1957 - WISHON DAM

FIG. 13 CONSTRUCTION VIEW ON NOV. 1, 1957 - WISHON DAM

FIG. 14 COMPLETION IN WINTER WEATHER - WISHON DAM

were observed. Due to the deeply jointed rock at Wishon many additional holes were necessary, and 26 of 300 grout holes took 100 to 700 sacks of cement.

Dumped Rockfill.—The spillway quarry and a main quarry near the right abutment, Fig. 7, furnished the rock for the dam. References (3), (6), (7) and (9) give a thorough description of quarrying, hauling and sluicing the rockfill. The powder factor was high, 1.0 lb. per cu. yd., due to deep fissures in the rock. Payment was made at a specified 50 cents a solid cubic yard for all material removed from the quarry site, including overburden. A bid price was paid for the yardage in the pay lines of the dam. The bid price included haul roads, hauling, sluicing, scarifying of lift surfaces and the additional cost of quarrying over the fixed payment. The fixed price is intentionally less than quarry cost. It gives both owner and contractor an interest in having minimum waste and reduces risk to the contractor if overburden or waste is excessive. Upon completion of the dam, it was computed that the dumped rockfill contains 32% voids. The rockfill at Courtright at the end of 1957 also was computed to contain 32% voids.

Placed Rock.—The placed rock production rate was greater than on previous dams of this type, averaging 135 and peaking at about 200 cu. yds. per 9-hour, 7-man crane shift. The reasons are: (a) simplified requirements of the specification; (b) the convenient work access for the operation of cranes and supplying of rock that is provided by the face lifts; (c) the use of very large, 6 to 15 ton, rocks; and (d) the well planned supply system and general organization of the contractor, Morrison-Knudsen Company. The large placed rocks were handled by a Lewis pin in a drilled hole or by two or sometimes three Sullivan pins in holes drilled at angles.[6,7]

Concrete.—The 3000 psi concrete mix contained aggregate in 4 sizes: # 4 to 3/8, 3/8 to 3/4, 3/4 to 1-1/2 and 1-1/2 to 2-1/2. The four rather than normal 3 sizes were used because of the friable nature of the aggregate and the importance of dense grading for maximum frost resistance. It was rescreened at the batching plant. The mix contained Permanente Type II cement, 15% Airox pozzolith, Possolith 3 (water reducing agent), and Dresinate XX (AEA) air entraining agent. A cement-pozzolan factor of 4.85 and slumps of 1 to 2 inches were used. Compressive strengths at 28 days were generally 3300 to 4000 psi with few cylinders below 3000. At 90 days tests exceeded 5000 psi. Concrete was mixed in a 1.5 cu. yd. paver on the dam and placed by crane and bucket from a face lift. Concrete slabs were poured very soon after the placed rock was placed, which is not theoretically ideal since the slabs must then take subsequent construction settlement. However, this scheduling makes convenient and minimum-cost construction possible, and the construction settlement is nominal and does not damage the slabs. Cold weather concrete was mixed with 120° to 150° water and the minimum temperature specifications were met with minimum air temperatures as low as zero Fahrenheit.[11]

Settlement of Concrete Face During Construction.—The early construction of face slabs subjects the slabs to settlement during construction. The lower construction cost with this practical scheduling is considered considerably more important than the reduction of slab settlement by delaying the pouring of face slabs. The dumping of the higher lifts causes settlement of the upper slabs that have been poured. However, the weight of the new dumped fill of the upper lifts does not have much effect on the fill underlying the slabs below the upper slab. The slabs further up in the dam and away from the more rigid

abutment slabs are subjected to the most construction settlement. In the several months after a slab is poured, the top settles downward more than the bottom. In fact, the bottom sometimes moves upward. This tends to cause compressive stresses in a line up the face and a closing of the soft horizontal joints. In general, settlement during construction was nominal and no cracks occurred in the concrete face. Some of the measurements that substantiate the above general comments will be summarized briefly and separately for the wing and main sections of the dam. Reference to Fig. 11 is helpful to see the stage of construction, and the location of settlement points on the dam are shown on the November 1 isometric of Fig. 11. Main Section construction settlement is shown on Fig. 15. The settlement of the face slabs during construction at Wishon is greater than that at Lower Bear River No. 1[10] due to the difference in scheduling and difference in face slopes.

Wing Section.—The wing section is 1500 feet long and generally of 100 to 150 foot height. Settlement points are on the corners of each slab, except for those located 15 feet from the cutoff line. The points 15 feet from the cutoff line (S points) and on the 6495 line (B points), about the center of the face, were set on September 1, 1957. Two months later, October 30, readings were taken again. Fifteen feet of fill and placed rock, and some concrete slabs, had been added, Fig. 11. The points 15 feet from the cutoff line moved downward vertically from zero to 0.04 feet. The B points moved "upward" from 0.00 to 0.06 feet and "upstream" 0.00 to 0.02 feet. Upward movement does actually occur. A possible explanation is that as the top fill is added, there is an upstream as well as downward movement of the rock underlying the upstream face, and since the face can't move upstream and downward without high compressive stresses, it resists and moves slightly upstream and upward. On May 13 of 1958, 6.5 months later and just before filling the reservoir, the S-points from Station 6 + 00 to 9 + 00 had moved downward 0.04 to 0.05 feet. During the same 6.5 months the B-points settled downward some 0.02 to 0.06 feet. All the movements during construction were very small and caused no cracks.

Main Section.—Some data on settlement of the concrete face of the main section during construction is presented in Fig. 15. During the two months of September and October 1957, Fig. 11, a 50 to 80-foot height of rockfill was added to the dam after the slabs between the C and D-lines had been poured. At the sections of maximum settlement, Station 21 + 60, the C point settled 0.68 feet while the D point settled 0.13 feet, Fig. 15. All settlement along the C line was downward during this two months whereas the ends of the D line moved "upward". The same characteristics may be observed for the B and C lines between the readings of October 30, 1957 and May 13, 1958.

Wishon Crest Settlement

Wishon dam was filled rapidly between May 9 and May 27. Four readings, three increments, of vertical crest settlement are plotted on a profile of the dam along the axis, Fig. 15. There is missing data at both ends of the dam because of limited time available to make a complete check of the survey data, at the time of completing this paper.

December 13, 1957 to April 21, 1958, Fig. 15.—All rockfill had been completed by December 13, 1957, Fig. 11, and the dam was being snowed in. Only a few top slabs had been poured so temporary crest settlement points were

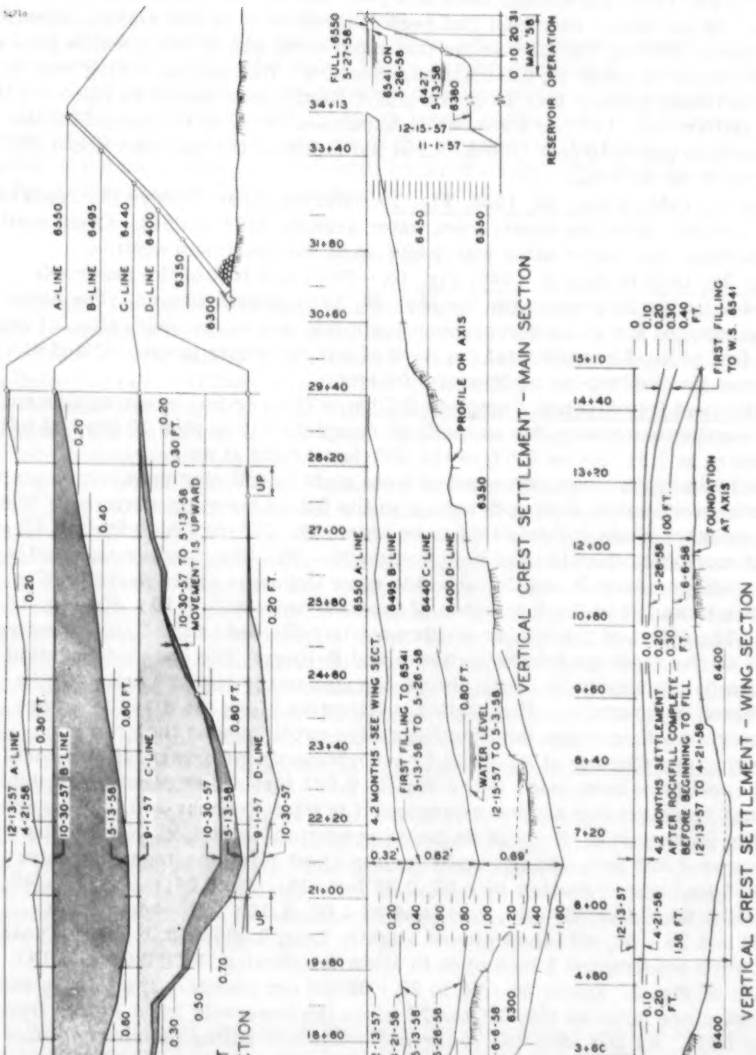


FIG. 15 FACE SETTLEMENT DURING CONSTRUCTION - 1ST FILLING CREST SETTLEMENT - WISHON DAM

set on the crest to record movement due to aging of the dam during the winter. On April 21, 1958, 4.2 months later and just before construction of the remaining 40 top slabs, the crest had been cleared of snow and measurements were taken. During the 4.2 months the water level was at 6380, which gave a 100 foot depth of water on the maximum section. This partial filling was to keep the intake grizzly free from ice and it should have had little effect on the crest settlement. For the 4.2 months, maximum vertical settlement on the wing section was 0.10 feet (158 ft. ht. at axis) and on the main section 0.32 feet (260 ft. ht. at axis).

May 13, 1958 to May 26, 1958, Fig. 15.—During these 13 days the reservoir filled to 9 feet from the crest, from water surface 6427 to 6541. Crest settlement between the above dates was 0.62 feet at the maximum section.

May 26, 1958 to June 6, 1958, Fig. 15.—The top 9 feet of the reservoir filled during the 24 hours after the May 26, 1958 measurements. The settlement measured for this interval represents that due to the application of the last 9 feet of head and the first ten days of full reservoir pressure, and at maximum section was an additional 0.69 feet.

Settlement Due to First Complete Filling.—The vertical crest settlement of the maximum section, due to the first complete filling plus 10 days of full reservoir, is 1.31 feet or 0.5% of the 260-foot height at axis.

The main section was constructed from right to left abutment, which gives maximum settlement, during dumping, to the fill on the right abutment. The crest measurements indicate this to be true. The 170-foot high Station 18 + 60 settled more than the 210-foot high Station 25 + 20. The construction settlement readings of the B and C-lines also show the same characteristic, Fig. 15.

There is a cliff in the foundation of the dam near Station 20 + 40, Fig. 15 and 1. The effect of the cliff on settlement is reflected in the C-line readings, but not on the readings for the higher A and B-lines. The abrupt foundation contours do not appear to cause abrupt changes in the higher portion of the face, which is favorable. The vertical settlement along the crest is a considerably smoother curve than the foundation profile under the crest.

Opening and Closing of Joints at Crest.—Measurements across the joints on the crest have been made every day (to 0.001 feet readings) since beginning of filling, to detect any sudden movement if it were to occur and to determine when the joints move. No joint on the wing section, to Station 14 + 40, changed as much as 0.005 feet. On the main section crest joint openings of greater than 0.01 feet were: Station 14 + 90, 0.03 feet; 15 + 10, 0.03; 0.015; 17 + 40, 0.025; 18 + 60, 0.015; 29 + 40, 0.04; and 30 + 00, 0.045. Between Station 20 + 40 and 25 + 80, all joints closed slightly from 0.005 to 0.02 feet. These joints were constructed 2 inch open to allow for closing, VERTICAL JOINT DETAIL of Fig. 2. Joints 26 + 40 to 28 + 80 did not change. The joint openings took place between May 20 and 24 while the reservoir level raised from 6490 to 6530. As and after the reservoir completed filling the opened joints began to close gradually, as much as 0.01 feet. The closing of joints took place later and more gradually than the opening, and all movement was in one direction. The joint movements are as would be predicted, and are favorably small.

The timing and nature of joint movements are consistent with previous measurements. The magnitude of the movement is on the order of the small movements at Lower Bear River Dams.(10) The opening of joints near the top of the dam and near the abutments occurs just before complete filling, when the major settlement normal to the face in the lower portion of the dam

is taking place. As the reservoir completely fills and the crest completes its initial settlement, the crest length shortens. This tends to close all joints, including those which previously opened.

Data and discussion has been confined to a brief presentation of the recently acquired crest settlement data. There are bronze pins in the corners of each slab. The measurements determine the locations of the pins in space, joint opening and closing, and change in length of the slabs. The change in length of the slabs gives a measure of the stresses in the slabs, and an indication of the movement of the rockfill under the slab. Points are also located on the downstream dumped rock face.

Courtright Dam

The site for Courtright Dam is a narrow granite canyon. Upstream is a very favorable reservoir site for a Sierra reservoir at such a high elevation, 8188 feet.(2) The dam is nine miles by road from Wishon Dam, and is located on Helms Creek, a tributary to the North Fork of the Kings.(2,8) Water released from Courtright goes to Wishon Reservoir. Annual carryover storage normally will be stored at Courtright, but the reservoir will be nearly or completely drained following a dry year.

The Courtright damsite had a narrow boulder filled gorge some 50 feet deep, Fig. 16. The left abutment was massive granite with "onion skin" surface, Fig. 20 and 21, and with talus of small rocks and fines. The right abutment was of blocky granite with jointing planes and had a talus containing many large rocks, Fig. 18. The choice of type of dam narrowed down to concrete arch and impervious face rockfill. Being a good arch site automatically eliminated a concrete gravity type. There being no impervious materials available eliminated consideration of a core rockfill type. The concrete face rockfill dam was adopted on the basis of estimated cost being 35% lower than that of the concrete arch dam. Estimates were rather reliable since Wishon rockfill dam and the Monticello and Donnells arch dams had only recently been bid. The Courtright profile is very similar to Monticello and Donnells profiles.

The design of Courtright is identical in many respects to that of Wishon and will be discussed only as it differs from Wishon. Comparative general data is presented in Table I. The data on Courtright Dam in Table I is different from previously published data. Between the 1957 and 1958 construction seasons it was decided to increase storage from 102,500 acre feet to 123,300 acre feet at the Courtright site.

Courtright Design

Diversion Tunnel and Outlet Works.—The diversion tunnel is located in the massive granite of the left abutment in preference to a shorter tunnel in the jointed rock of the right abutment, Fig. 16. The alignment passes under the end of the dam in order to permit the installation of a Stevens water surface detector in a 4 inch drill hole from crest to tunnel. The submerged tower is similar to that at Wishon.(8) An underground valve chamber, similar to that of Wishon,(8) houses a 72 inch butterfly valve and a 42 inch Howell-Bunger valve. The tunnel is unlined for the full length. The sizes, Fig. 16, were determined by permanent outlet requirements and not temporary diversion requirements, since the drainage area is small and the design permits relatively

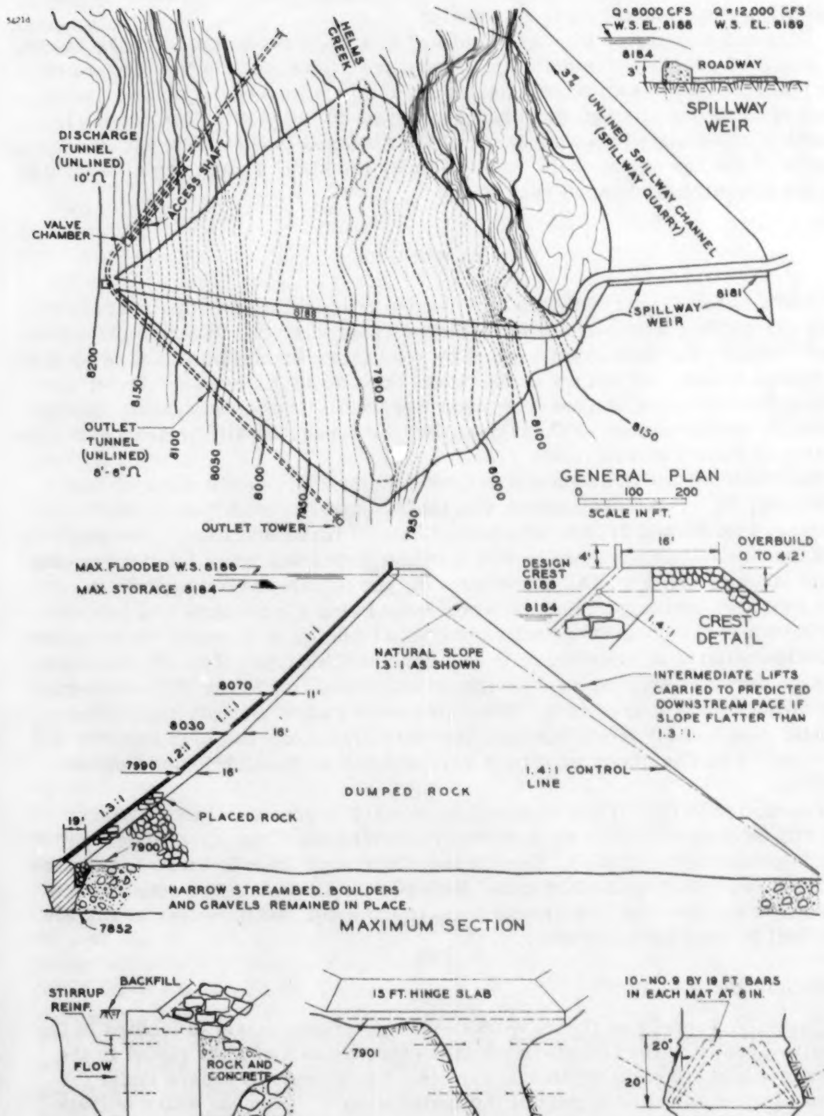


FIG. 16 PLAN-SECTION-CUTOFF PLUG-COURTRIGHT DAM

small flood flows to pass through the partially completed rockfill. The valve chamber also contains a 2 foot diameter bypass with two gate valves for draining the reservoir in the unlikely event of trouble with the main outlet.

Spillway.—The right abutment quarry provides a long open crest spillway that will pass an 8000 cfs design flood with only 4 feet of depth and with the 4 foot coping as freeboard, Fig. 16. The unlined discharge channel is designed to carry increased flow as the reservoir water surface encroaches upon the freeboard of the coping wall. This gives additional security at no cost. The road is located in the spillway channel since reservoir operation is such that spilling nearly always may be prevented if necessary. An ungated spillway is economic at Courtright, whereas it was not at Wishon because of the smaller spillway capacity and the lower incremental cost of a foot of height of dam at the Courtright site.

Cross Section.—The 1.4:1 control line for the downstream face slope intersects the upstream face slope at the design crest elevation, Fig. 16. This means that nearly the full height of the downstream slope will be at the natural dumped slope. The crest width of 16 feet is considered a practicable minimum for construction and for such a high dam. The 20 foot crests at Lower Bear River and Wishon were to provide two-way roads and permit passage over the crest when maintenance equipment was parked on the crest. These requirements were not imposed at Courtright.

Alignment.—The design axis of the dam is along a segment of a 5000 foot radius curve at design crest elevation. The layout of the dam is from a chord, called the baseline, connecting the intersections of the axis and the abutments. The vertical joints are in planes perpendicular to the baseline, Fig. 17. At the crest the middle ordinate is 18 feet and at horizontal joint 8030 it is 3 feet. In the lower area of the face the predicted settlement will nearly change the moderate upstream arching to a straight line. At the crest enough curvature will always remain to give a favorable appearance. The dam is cambered with 4.2 foot overbuild at the maximum section.

Streambed Cutoff Plug.—The streambed material consisted of boulders tightly embedded in sand and gravel. It was considered a suitable foundation for a rockfill but was excavated to provide a cutoff for the dam. The streambed cutoff plug, Fig. 16, was designed as a shear plug with a shear stress of 150 psi. Near the top of the plug, stirrups were used.

Cutoff and Grouting.—The left abutment presents no cutoff or grouting problems since the granite is massive and without seams other than the "onion skin" near the surface, which was removed. The right abutment, with its deep jointing planes, requires special consideration and a careful and thorough grouting program.

Concrete Face.—The only difference from the face at Wishon is the joint arrangement, Fig. 17. The sloping joint is used for the full perimeter to accommodate the high differential movements that are anticipated due to the steep abutments. The right abutment is so steep that several sloping joints have been used. Only a short distance out on the face from the right abutment cliff, the water pressure is carried through a long distance in rockfill to the foundation. High settlement will occur close to the fixed and unyielding cutoff wall and the several hinged slabs should tend to prevent cracks. Between the slab and its seat on the cutoff, Fig. 3, a 2-inch strip of styrofoam is used for the full length of cutoff wall to permit the slab to move at the cutoff line.

The design, as it relates to foundation excavation, can best be reviewed along with comments on construction. The dumped and placed rockfill

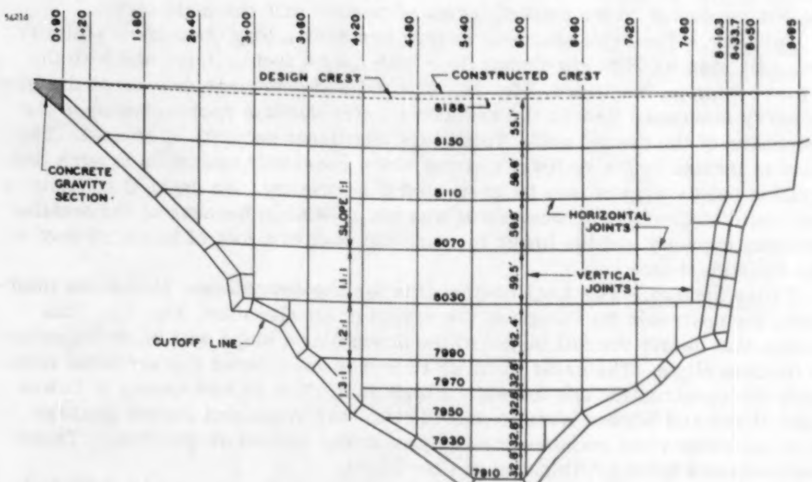


FIG. 17 DEVELOPED VIEW OF FACE - COURTRIGHT DAM

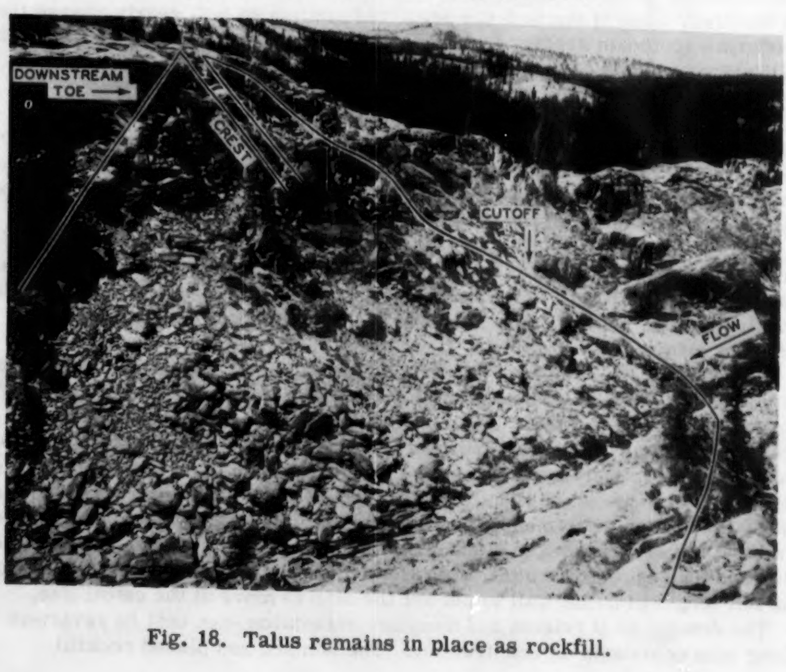


Fig. 18. Talus remains in place as rockfill.

specifications and methods are the same except that sluicing is at a water to rock volume ratio of 2:1 rather than 3:1. This change from Wishon was made since clean rock was expected and there is not quite enough summer water supply at Courtright, for the 3:1 ratio, without recirculating.

Courtright Construction

A nine-mile road was constructed from Wishon Dam to the Courtright damsite in late 1955. The contract for the dam was awarded to Morrison-Walsh-Perini in December 1955. In four months of 1956, July through October, construction consisted of clearing, diversion tunnel, streambed plug, excavation, cutoff and opening of quarry. By asphalt paving the road in 1956, and keeping it snow-plowed during the winter, an early start in May of 1957 gave a longer than normal construction season in 1957. Six months of construction in 1957 brought the dam to the stage shown on Figs. 19B and 21. Due to very heavy snows in the winter of 1957, construction was not underway until early June of 1958, and then in six to eight feet of snow. With favorable weather at the end of the 1958 construction season, it may be expected that the dam will be completed in 1958.

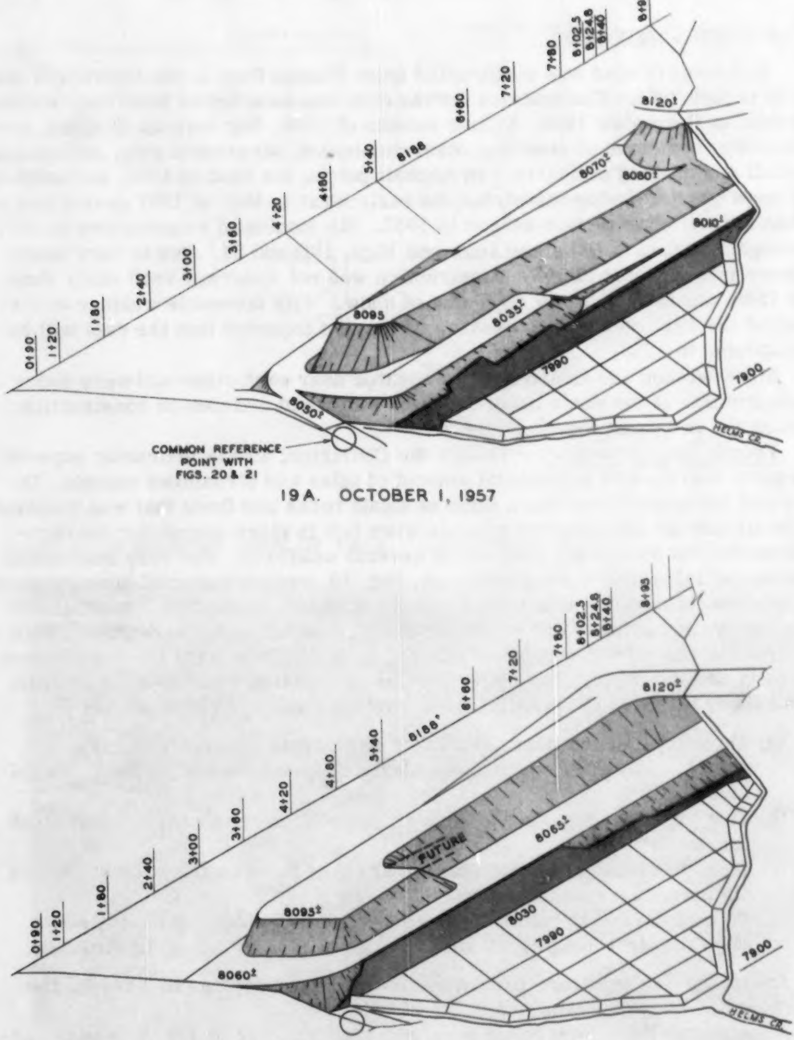
Since Wishon and Courtright are located near each other and were under construction at the same time, the prior published articles on construction cover the two dams together.[1,3-7,9]

Foundation Excavation.—Though the Courtright site is primarily exposed granite, there was a substantial amount of talus and streambed deposit. On the left abutment there was a talus of small rocks and fines that was removed. The streambed boulders and gravels were left in place except for the excavation for the streambed plug and of several sandbars. The very substantial volume of talus on the right abutment, Fig. 18, required special consideration. It was considered that this talus probably would be as good as a rockfill and the cost of removing it and replacing it with rockfill could be avoided. Five drill holes indicated the depth of talus to be 30 to 50 feet and the composition to be rocks and earth. The major portion of the talus was downstream from the axis of the dam. It was decided to leave the talus in place subject to:

(a) Excavation of the talus to 150 feet downstream from the cutoff to assure knowledge of conditions underlying the lower half of the concrete face.
(b) To reconsider leaving the talus in place after seeing the excavation of (a).
(c) To hydraulic monitor the surface areas of fines on the talus slope at a bid price per monitor-hour.
(d) To explore and to remove some fines from the top of the talus slope when the first dumped fill lift provided access to the top of the talus.

Except for the arbitrary excavation (a) and some pockets of fines (d) the talus remained in the dam.

A suggested lift arrangement with face lifts, similar to Fig. 5, was included in the bidders' set of drawings. The contractor, Morrison-Walsh-Perini, improved on the suggested arrangement by ramping access roads to the face lifts within the rockfill to avoid the cost of excavating haul roads in the solid rock abutments. The actual arrangement is illustrated by the isometrics of Fig. 19 and photos of Figs. 20 and 21. A common point on each view is circled. Fig. 20 shows the stage of construction on August 21 when the



19 A. OCTOBER 1, 1957

19 B. NOVEMBER 1, 1957
(END OF 1957 CONSTRUCTION SEASON)

FIG.19 ISOMETRIC VIEWS OF CONSTRUCTION – COURTRIGHT DAM

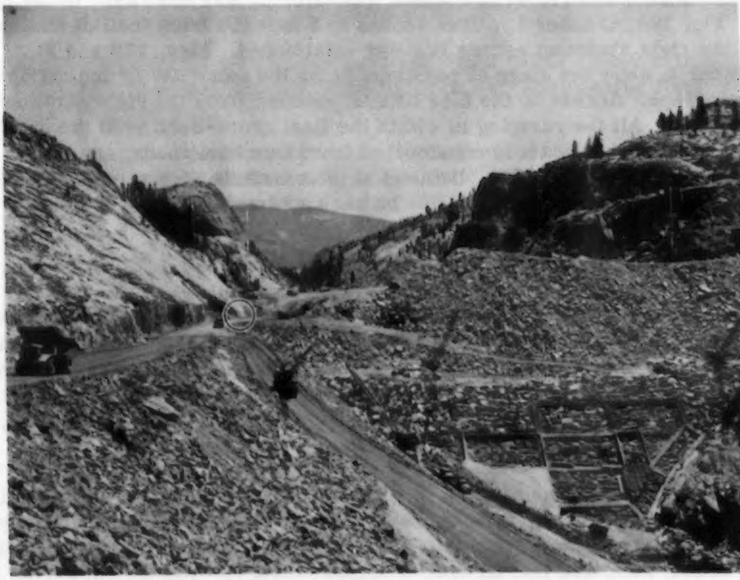


Fig. 20

Fig. 21. Kings River Project. A new lift has been started in the background in the construction of Courtright Dam. The cutoff trench carved into the left abutment beyond the crane near the edge of the dam in the background indicates the height still to be reached.

dumped rockfill was rising in a spiral and ramping was downward to the face lifts. Fig. 19A, October 1, shows access to a face lift from the left abutment while the right abutment access is being established. Figs. 19B and 21, November 1, show the stage of construction on the same day by isometric and by photograph. Access to the face lifts is possible from the right abutment access road. All the ramping is within the final cross-section of the dam. The 317-foot rockfill dam is constructed from four haul roads; one near streambed level, one on each abutment at intermediate levels, and one at the crest. Two quarries were used, one on each abutment.

CONCLUSION

The design included a number of features to reduce costs over previous designs, and to make the construction of these dams a continuous and straightforward production job. Economies beyond the design changes have been made by the contractor for both dams. Their effective use of face lifts, methods in supplying and handling placed rock, use of exceptionally large placed rock, and overall job planning has kept their costs down and brought both dams well ahead of schedule.

Features were incorporated in the design and construction of the placed rock and concrete face, to effect substantial savings in cost, that knowingly subjected the face to some possible cracking. Observations, during and after construction, indicated that no cracks occurred and that settlement during construction was nominal. The concrete face and placed rock may be considered as the impervious membrane for the dam, and are treated in detail in this paper. However, the most important and main structural element of the dam is the well sluiced dumped rockfill.

Careful measurements are being kept of the movements of both dams and should contribute to future dams of greater height, still lower costs, and of unquestioned safety.

ORGANIZATION

Engineering was under the direction of Walter Dreyer, vice president and chief engineer of Pacific Gas and Electric Company, and H. V. Lutge, then chief civil engineer. I. C. Steele was consulting engineer. The experience of T. J. Corwin, supervising civil engineer, has been particularly helpful. The writer is indebted to J. E. Schumann, A. Seklemian, and A. G. Strassburger, who have been engaged with him on the engineering studies and design. The help in the review of this paper by Walter Dreyer and by J. D. Worthington, chief civil engineer, is gratefully acknowledged.

Construction is under the direction of A. J. Swank, vice president in charge of general construction, and H. W. Haberkorn, manager of hydroelectric construction. Joe Pirtz, Jr. is project superintendent and George Thacher, project engineer.

The contractor for both dams is Morrison-Walsh-Perini, sponsored by Morrison-Knudsen Company and represented by Jim Wells, vice president. B. L. Perkins is project manager and John F. Erdle is project engineer.

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Paper 1747 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: THE PARADELA CONCRETE FACE DAM

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and Nuno de Vasconcelos Porto[3]
(Proc. Paper 1747)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

ABSTRACT

The 361-foot high Paradela Concrete Face Rockfill Dam forms part of the hydro-power development of Northern Portugal. The paper discusses the selection of site and type of dam, describes the design and the construction details and procedure, and presents performance data.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1747 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

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General Considerations

The hydro-electric development of the River Cávado and its tributary the Rabagão, of which the Hidro Eléctrica do Cávado, S.A.R.L. is the concessionary, is situated in the North of Portugal in a district where the drainage area of those rivers has an average annual rainfall of about 2.100 mm (82.7 in.). (See Fig. 1)

The utilised differences of water level lie between the elevations 903 m. (El. 2,963 ft.) for the Cávado River, 888,7 m. (El. 2,916 ft.) for the Rabagão River and 41 m. (El. 135 ft.), the corresponding gross heads being 862,0 m. (2,828 ft.) and 847,7 m. (2,781 ft.), respectively.

The hydrological system of Portugal, which is practically uniform all over the country with generally low flows in summer time, as well as the scarcity of coal and its poor quality, gave rise to the idea of a development scheme capable of providing a high water storage. Accordingly, a number of storage developments were planned culminating in the Alto Rabagão project, whose big reservoir, in spite of its relatively small drainage area, fully justifies itself economically. In fact, the 781 x 10^6 m3. (633,000 acre. ft.) storage of the Alto Rabagão reservoir provides for a storage in energy of 1.364 x 10^6 kWh. taking into consideration, of course, the heads of the downstream developments. So the Portuguese grid system will have at its disposal a considerable amount of energy in reserve to provide for possible dry years, a fact of greater importance if we remember that the total Portuguese demand in 1964, the year in which the project will be concluded, will be about 4.3 x 10^9 kWh.

The general characteristics of the various projects of this scheme are shown in Table 1.

The Paradela Project

The Paradela project was the fourth carried out by the Hidro Eléctrica do Cávado. (See Fig. 2)

This is a development with a dumped rock-fill dam in the valley of the Cávado creating a reservoir with a capacity of 158 x 10^6 m3. (128,000 acre. ft.) and an area of 380 ha. (939 acres). The water is diverted to an unreinforced concrete lined pressure tunnel which is only steel lined in certain sections, either where there are bad rock characteristics or where the head is greater than 110 m. (361 ft.). The inside diameter of this tunnel varies from 2,80 m. (9.2 ft.) to 2,25 m. (7.4 ft.) and its total length is 9,5 km. (31,170 ft.). Near the reservoir there is a caterpillar gate with 2,50 x 3,80 m. (8.2 x 12.5 ft.) and a steel trash rack operated in an intake shaft. The tunnel intake has a fixed reinforced concrete trash rack.

Near the end of the underground works there is a butterfly valve placed in a chamber, to protect the open air penstock. Upstream from this valve is located a shaft surge tank with compensation galleries and an open air expansion tank.

The penstock layout has a slightly inclined section, amounting to 1.350 m. (4,429 ft.) followed by a 570 m. (1,870 ft.) steeply inclined section. Its diameter varies from 2,35 m. (7.7 ft.) to 2,05 m. (6.7 ft.). It has no expansion joints and its variations in length due to temperature changes are made possible by the distortion of the penstock at the bends; in the straight sections, the tendency to expand or contract is taken up by the internal stress in the pipe.

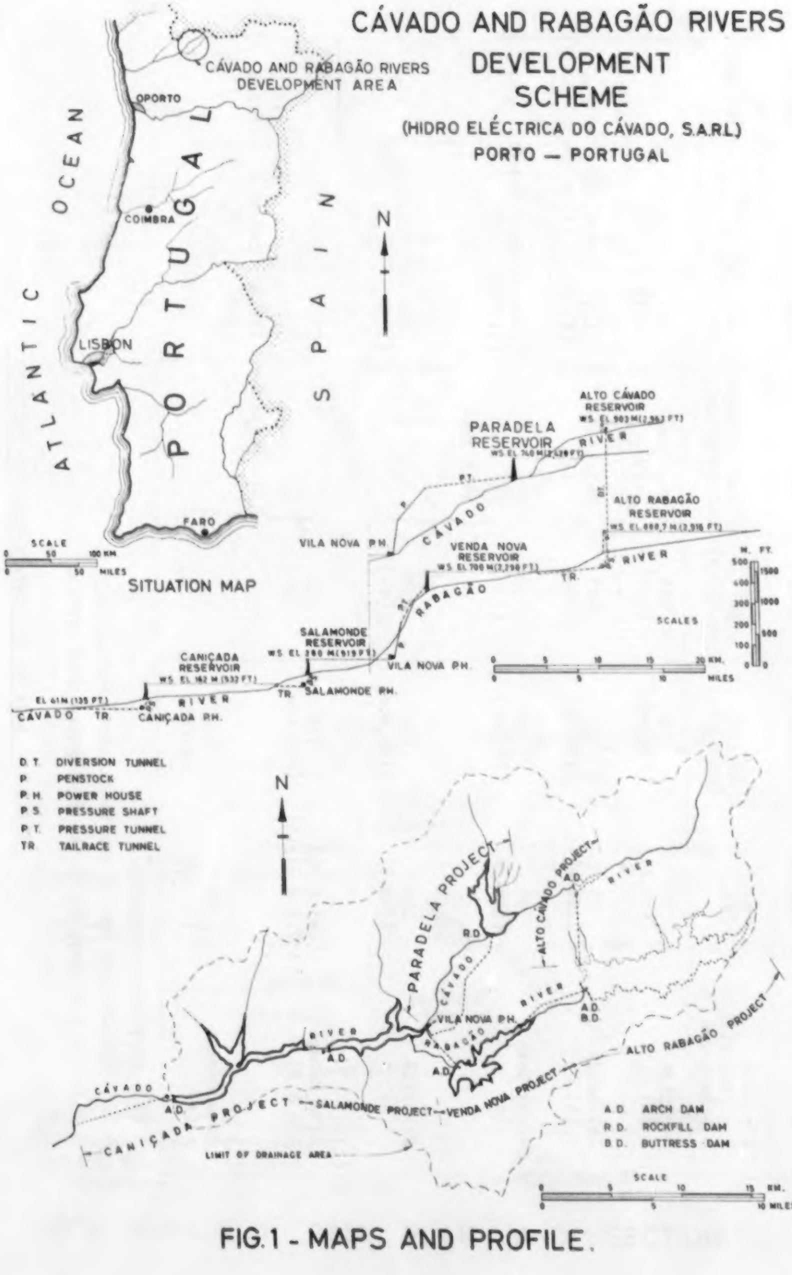


FIG. 1 - MAPS AND PROFILE.

Dam	Maximum heigth	Effective capacity	Drainage area	Power House	Static head	Units	Annual output
A - Arch R - Rock-fill B - Buttress	m. (ft.)	10^6 m3. (acre.ft.)	km^2 (sq.miles)	C-Conventional U-Underground	m. (ft.)	kVA F-Francis P-Pelton	10^6 kWh
Caniçada (A)	76 (249.4)	138 (112,000)	783 (302)	Caniçada (U)	121 (397)	2x32.000 (F)	260
Salamonde (A)	75 (246.0)	55 (45,000)	623 (241)	Salamonde (U)	127 (417)	2x25.000 (F)	200
Venda Nova (A)	97 (318.3)	92 (75,000)	342 (132)	Vila Nova (C)	414 (1,358)	3x30.000 (P)	200
PARADELA (R)	110 (360.9)	158 (128,000)	126 (49)		460 (1,509)	(2) 60.000 (F)	260
(1) Alto Rabagão (A & B)	(103 & 65) (337.9 & 213.3)	776 (629,000)	210 (81)	Alto Rabagão (U)	192,8 (784)	2x40.000 (F)	(3) 1.200
(1) Alto Cávado (A)	98 (91.9)	5 (4,000)	102 (39)				

(1)- Proposals requested.

(2)- Can utilise one of the Venda Nova units.

(3)- Dry year support to the Portuguese grid system.

TABLE 1 - DATA ON THE CÁVADO-RABAGÃO PROJECTS

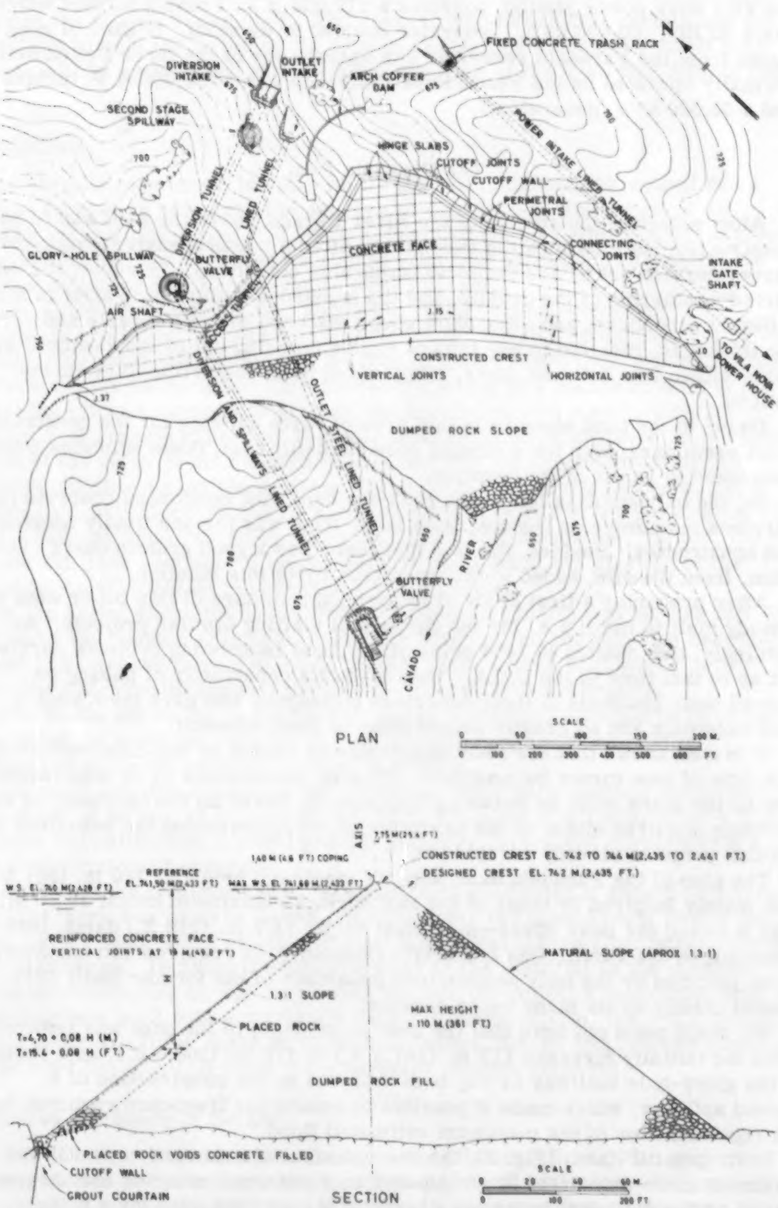


FIG. 2 - PARADELA DAM. PLAN AND SECTION.

The water, after passing through an in-line self-closing valve erected in the Vila Nova power station, operates a 117.000 H.P. Francis turbine which has a 50 Htz., 60.000 kVA., generator coupled to its shaft. In case of need the water from the Paradela reservoir can operate one of the three Pelton sets normally operated by the Venda Nova reservoir water (39.000 H.P. turbine and a 30.000 kVA. generator.)

The Paradela Dam

After selecting the stretch of the River Cávado where the dam was to be constructed, its final location was chosen after a comprehensive geological survey carried out to find the most favourable conditions. However, the very varied weathering of the granite, and the numerous faults which exist in many different directions, and often show gouge material of considerable and varying thickness, resulted in any type of rigid dam being out of the question, even those which, like buttress dams, might stand more distortion of the foundations.

Owing to the total absence in this area of earth suitable for the construction of an earth dam, even for a sloping core rock-fill dam, these solutions were also found to be out of the question.

So, the only solution left was a rock-fill dam with reinforced concrete impervious membrane on the upstream face. This was the one finally adopted and constructed. Besides, the fact that there was a good granite quarry about 2 km. from the site, added to the practicability of this solution.

After preparing a first study of the dam, the authors of this paper went on a study visit to the U.S.A. for the purpose of visiting similar projects. Accordingly, they visited in 1954 some of the most interesting projects carried out up to that time in the U.S.A. They take this opportunity of putting on record their gratitude to their American colleagues who gave them such a kind reception and so greatly helped them on their mission.

It is well known that the countless problems raised by the construction of this type of dam cannot be mastered either by calculations or by experiment. And so the plans must be drawn up empirically, based on the teachings of experience acquired either in the execution of, or in observing the behaviour of similar projects already carried out.

The plan of the Paradela dam, with its maximum height of 110 m. (361 ft.), was mainly inspired by those of the Salt Springs—maximum height 100.0 m. (328 ft.)—and the Bear River—maximum height 73,2 m. (240 ft.) dams, both belonging to the Pacific Gas & Electric Company; the required extrapolation being justified by the fully satisfactory behaviour of the former (built 1931), proved chiefly by its many years service.

We would point out here that the maximum height of the dam was lowered from the initially foreseen 112 m. (367.5 ft.) to 110 m. (360.9 ft.), as a result of the glory-hole spillway having been relieved by the construction of a second spillway, which made it possible to reduce the freeboard required for the regularization of the maximum estimated flood.

In its general lines, (Fig. 2), the dam consists of a dumped rock-fill embankment made watertight by an impervious reinforced concrete membrane placed on the upstream face over a layer of placed rock used for a better distribution of the reaction of the dumped rock-fill on the membrane.

The seepage through the foundation is prevented by means of a grout curtain of cement and silicates varying in depth according to the hydrostatic load and the quality of the rock. The connection between this curtain and the watertight membrane covering the face is secured by a concrete cutoff wall poured into an open trench running along the base of the upstream face of the dam.

Foundations

As already mentioned, the site where the Paradela Dam is located has a granite foundation with variable degrees of weathering. By means of extensive excavations—about 370.000 m3. (480,000 cu. yd.)—the foundation area was regularized and the most weathered rock was extracted, although some patches of rather weathered granite were still left.

The following treatment was applied to these patches which might be eventually eroded by leakages from the curtain: After a cleansing by a jet of water, the ground was covered by a layer of hard granite over which the rock-fill would later be dumped. This procedure aimed at a double effect. In the first place it was intended to provide the foundation with a protection layer against soil erosion caused by eventual leakages; this is achieved by the filling in of the voids in the crushed rock layer with the fines carried by the copious sluicing during the dumping of the rock-fill. In the second place, it lessens the settlement of the rock-fill where this is due to the penetrating of rock points or small surfaces into a relatively soft foundation. In fact, the stones will then rest upon a larger surface obtained at the expense of the distribution afforded by the crushed granite cushion.

In the areas where the foundation of weathered granite was sloping and did not allow the cushion granite to remain in position, all the more so because it had to resist the impact of great blocks, a dry rubble wall had to be constructed which would support the steeply sloping crushed granite.

The faults appearing in the surface of the foundation also had to be properly treated, to avoid the erosion of their granite gouge and their more weathered edges by leakages through the banks.

To carry out this treatment the faults and their surroundings were cleaned to a depth of 60 cm. (2 ft.) and the material taken out was replaced by a filter of sand and calibrated crushed granite placed in layers and covered with a layer of "cyclopic" concrete.

As far as the treatment of the foundation is concerned it should be pointed out that there was an area on the left bank, near the downstream foot of the dam, where the granite exists in almost horizontal layers alternatingly hard and weathered. To avoid the washing away of the material in the layers of greatest weathering by waters coming through the banks, a supporting wall of concrete was built with a maximum height of about 10,0 m. (32.8 ft.), which could support a filter made up of various layers of sand and crushed granite to prevent the smallest fines from escaping into the rock-fill.

Location and Shape of the Dam

The location of the dam was fundamentally in accordance with geological requirements because it was decided that the cutoff wall should be located in an area of less weathered rock. Even so, it was necessary to excavate at some places to a depth of 15 m. (49.2 ft.).

In plan, the dam is in the form of a curve. (Fig. 2) This curvature is intended to produce, by means of the arch action, the following results: to oppose the opening of the vertical joints and the tendency that has been verified for the rock-fill to move towards the centre of the valley. Accordingly, the dam is defined by a horizontal directrix, formed by two branches of hyperbolas tangent at their vertices which coincide with joint 15. The hyperbola was chosen because it is a curve concentrating the curvature near its vertex and it was important to have more curvature in the central and lower part of the dam where the distance between the banks is less.

The vertex of the hyperbola was fixed at joint 15 because the greatest settlements were expected at the profile of this joint, for although the profile of the dam is not greater in height on the upstream side, it is in its vertical plane that we find the longest "columns" of rock-fill that will have to transfer the water pressure to the foundation.

In profile (Fig. 2) the upstream face of the placed rock was defined with a uniform inclination of 1,3/1 (1,3 in the horizontal and 1 in the vertical) and the downstream face would have the natural slope of the rock-fill, and this, moreover, was found to be of practically the same value as that assumed above. In consideration of the extremely slight seismic activity of the region and the expected good loose rock-fill dumping, no objection was seen to leaving the downstream face with the natural rock-fill slope, so much so that with the width planned for the crest and with the extra height of 2,00 m. (6.6 ft.) anticipated in the central and highest part of the dam—in expectation of the vertical settlement—the width of the base corresponding to the downstream slope, commonly planned at 1,4/1, was practically attained.

On the other hand, as we expected, the safety of the downstream toe of the dam is increased as a result of the process of construction adopted. In reality the dumping of lifts from a very great height causes a lessening of the slope of the lower part because of the great distance that the big rocks of the rock-fill roll after the fall from such a great height during which they have acquired considerable kinetic energy.

The crest was planned with 7,75 m. (26.4 ft.) width, 0,75 m. (2.5 ft.) being for sidewalk on the upstream side, 6,00 m. (19.7 ft.) for roadway and 1,0 m. (3.3 ft.) for berme on the downstream side.

Following the example of the Bear River Dam No. 1 the profile on the upstream side was not made concave because the distribution of the settlements of the rock-fill under the action of water pressure would provide this advantageous curvature.

The Placed Rock

A great deal of interest was attached to the placed rock layer to be introduced between the loose rockfill mass and the reinforced concrete face on the upstream side. The excellent workmanship available in Portugal made fine results possible in this kind of work. (Figs. 9 and 10).

Considering the height of the dam, it was thought that this layer should be thick, so that the reactions of the loose rock-fill should be well distributed, and of variable thickness increasing with the depth according to the law:

$$e_1 = 3{,}00 + 0{,}05\, h_1 \qquad (e_1,\ h_1 \text{ - meters})$$

or:

$$e_2 = 10 \quad + 0.05\, h_2 \qquad (e_2,\ h_2 \text{ - feet})$$

FIG. 9 — CRANES PLACING ROCK (STAGE 1).

FIG. 10 — CRANE PLACED ROCK. DETAIL.

in which "h" is the distance to the crest and "e" the thickness measured in the normal to the face. Hence, we have, at the lowest level of the dam, a layer of 8,60 m. (28.2 ft.) or, measured horizontally, approximately 14,00 m. (45.9 ft.).

Where the placed rock-fill comes in contact with the foundation, that is, near the cutoff wall, a special layer, 1,00 m. (3.3 ft.) thick, was made in which the voids between the rocks were filled with concrete. It was intended, by this means, to get a better distribution on the ground of the pressures transmitted by the concrete face while the foundation was at the same time protected, in the neighbourhood of the connection of the concrete face with the cutoff wall, from the erosive action of eventual leakages, the energy of which is not lost by passing through the placed rock. At the same time, it prevents the grout from escaping into the rock-fill in the neighbourhood of the cutoff wall, during the execution of the grout curtain.

The Cutoff Wall

As stated before, the cutoff wall ensures the connection between the grout curtain and the concrete face. Besides, it prevents the leakages which, under the water load, might take place either through the surface on the foundation or through soil immediately below the latter.

Given the characteristics of the foundations, it was thought necessary to provide for an inspection gallery in the body of the cutoff wall along almost the whole of its length. (Fig. 3) It is thus possible to control by means of conveniently placed drains the behaviour of the foundation grout curtain, as well as to carry out repairs to the said curtain, even with a full reservoir. Owing to the very nature of rock-fill dams it would be impossible to carry out such inspections in any other way.

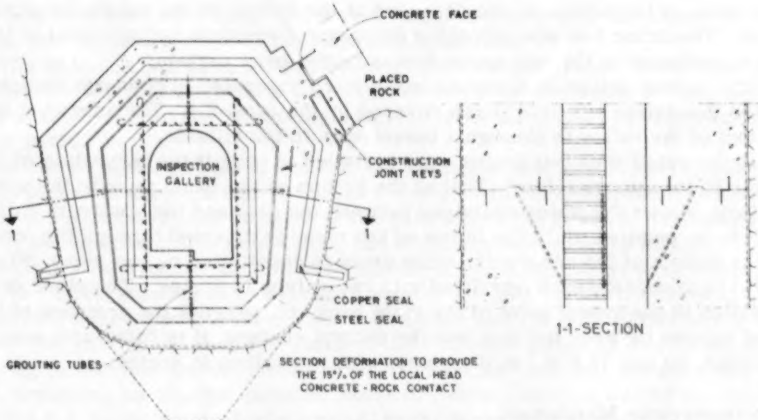
The necessary "gabarit" for the work in the gallery required that its cross-section should have 2,50 m. (8.2 ft.) in height and 1,70 m. (5.6 ft.) in width.

It was also decided that the minimum thickness of the concrete between the gallery walls and the foundation or the reservoir water should vary, according to the elevation, between 1,85 m. (6.1 ft.) and 2,00 m. (6.6 ft.).

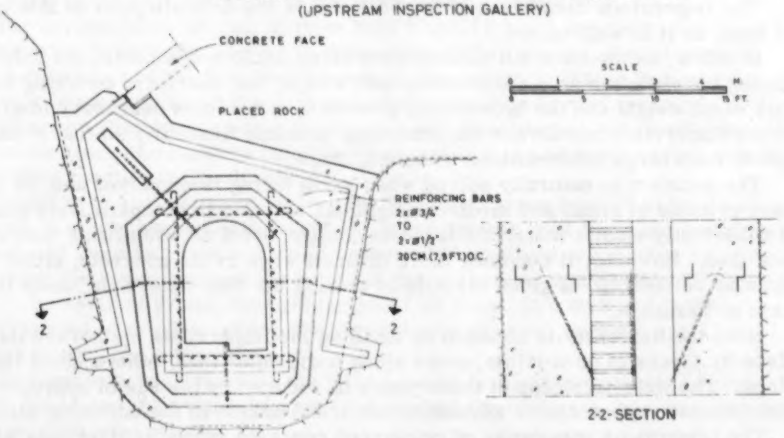
On the other hand, it was determined that the development of the concrete rock contact line should be at least equal to 15% of the head corresponding to the water load in the section, so as to provide a sufficient length for the water percolation.

In the high parts of both banks, and so as to economize on excavations, mainly on the left bank where they reached a depth of 15 m. (49.2 ft.), the location of the cutoff wall was moved downstream and it was constructed over the placed rock. In the final part of the right bank, between joints 33 and 37, the cutoff wall was designed without inspection gallery as it was a part under slight pressure and easily accessible as a result of the normal operation of the reservoir.

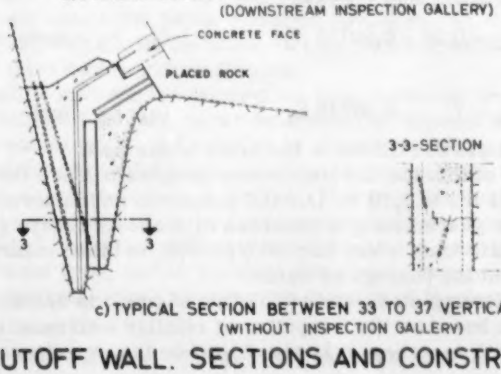
Between consecutive connecting joints of the face the cutoff wall was poured in two blocks to enable the concrete to contract without structural damage. The joints between these blocks, in addition to the connecting keys, were fitted with water seals, made of sheet copper on the outer perimeter and of mild steel sheets on the perimeter of the inspection gallery. Compartments were thus formed which could be filled with cement grout and so watertightness was guaranteed.



a) TYPICAL SECTION BETWEEN 6 TO 29 VERTICAL JOINTS.
(UPSTREAM INSPECTION GALLERY)



b) TYPICAL SECTION BETWEEN 0 TO 6 AND 29 TO 33 VERTICAL JOINTS.
(DOWNSTREAM INSPECTION GALLERY)



c) TYPICAL SECTION BETWEEN 33 TO 37 VERTICAL JOINTS.
(WITHOUT INSPECTION GALLERY)

FIG. 3 - CUTOFF WALL. SECTIONS AND CONSTRUCTION JOINTS.

The inspection gallery of the cutoff wall has 3 accesses: at the top of the left bank, at the bottom of the dam, and at the bottom of the col on the right bank. The latter two also allow the drainage of eventual leakages and of the water collected in the inspection drains in the grout curtain.

The access galleries from the banks were concreted in open-air trenches in the foundation and afterwards covered by the rock-fill. The access at the bottom of the valley is through a tunnel open in the hillside.

In the cutoff wall two drains were installed to permit the inspection of the whole of the concrete face. That at the bottom of the dam, close to the access gallery, allows the water contained between the dam and the upstream coffer dam to be emptied while the inflow of the river is diverted through the outlet at the bottom of the reservoir. This drain is constituted by two pipes, 30 cm. (1 ft.) in diameter, each one fitted with two valves in series. The other drain, installed in the lowest point of the right bank col, permits the drainage of the dead volume between the dam and the natural surface; it is fitted with only one pipe, 20 cm. (7.8 in.) in diameter, with two valves in series.

The Impervious Membrane

The impervious membrane of the rock-fill is the delicate point of this type of dam, as it is well known.

In effect, as the rock-fill is subject to large scale settlements, not only during construction as a result of its own weight, but also later on owing to this same weight and the hydrostatic pressure, it becomes necessary to give to the impervious membrane the maximum possible flexibility so that it can follow such large deformations without damage.

The problem is naturally solved when earth facing is employed and, in the case of dams of small and medium height, its solution is comparatively simple if sufficiently elastic materials (such as timber, steel or bituminous concrete) are used. However, it becomes more difficult when to the concrete, either plain or reinforced, is given the role of making the dam watertight, as is the case at Paradela.

Here the flexibility is obtained by dividing the impervious membrane into slabs by means of open joints, some along horizontal lines, others along the slope. The watertightness of these joints is secured by the use of appropriately shaped copper sheets embedded into the concrete of the adjoining slabs.

The impervious membrane of reinforced concrete varies in thickness with the depth, in accordance with the law:

$$e_3 = 0{,}30 + 0{,}00735\, h_3 \qquad (e_3,\ h_3 \text{ - meters})$$

or:

$$e_4 = 1 \quad + 0.00735\, h_4 \qquad (e_4,\ h_4 \text{ - feet})$$

where "h" is the depth in relation to the crest of the dam.

In consequence of this law the impervious membrane has a thickness varying from 0,30 m. (1 ft.) to 1,10 m. (3.6 ft.) measured on the normal to the concrete face, thus guaranteeing a thickness of concrete always greater than 1% of the hydrostatic load, which has been proved, in other enterprises, to be sufficient to prevent the passage of water.

As for the reinforcing steel, the calculation of which is necessarily fallible because it must be based on the computing of relative settlements which cannot be predicted with accuracy, it is also based on the experience acquired in

similar enterprises. This experience has shown that a mesh of density corresponding to a section of 0,5% of the concrete section, in both directions (horizontal and in the direction of the slope), is sufficient.

This is how it has been done at Paradela, the reinforcement being placed in one or two layers according to the thickness of the concrete, 1" bars being used at the lower levels (where the concrete is thicker) and 3/4" bars in the upper parts.

As can be seen in Figs. 4, 6 and 7, the Paradela dam has 14 horizontal joints placed at about 10 and 15 m. (32.9 and 49.3 ft.) interval, the shortest being in the lower and central parts of the dam where the greatest settlements are to be expected. The joints along the slope are spaced at 15 m. (49.3 ft.). In the same figures can also be seen details of the different types of joints.

The copper sheet used to seal the joints is 1,5 mm. (0.059 in.) thick in those below elevation 690.50 m. (El. 2,265.5 ft.) and 1,0 mm. (0.039 in.) in the rest. The part of the seal embedded in the concrete varies in dimension according to the joint and its elevation: 274 mm. (10.7 in.) and 224 mm. (8.7 in.), depending on whether situated above or below elevation 690,50 m. (El. 2,265.5 ft.), in the vertical joints;* and, in the horizontal joints, 281 mm. (11 in.) when below that elevation and 231 mm. (9 in.) when above.

The horizontal joints, all of them with 3 cm. (1.2 in.) openings, are filled with timber boards that, owing to their great deformability, absorb part of the deformation of the face which tends to close such joints; in this way an attempt is made to avoid the crushing of the concrete around these joints (a phenomenon observed in some facings already constructed and tried), and, at the same time, the relative rotation of adjoining slabs is made easier without the above mentioned disadvantage.

In order to avoid squeezing the groove of the copper seal when the joint closes, a steam packed cork board is placed against, and glued to, one of its sides. At the other side a strip of "rubberoid" prevents the mortar from entering the space between the seal and the timber, as can be seen in Fig. 4.

In the vertical joints, the opening has 7 or 5 cm. (2.7 or 2 in.) in the central part of the dam where the curvature is greatest, and 2,5 cm. (1 in.) in the lateral and lower parts.

Downstream from the seal the joint opening is filled with steam packed cork board and the groove is protected against squeezing in the same way as in the horizontal joints.

Upstream from the seal the opening widens to 20 and 15 cm. (7.8 and 5.9 in.), respectively above and below elevation 690,50 m. (El. 2,265.5 ft.), and this widening may eventually be filled, in case of need, with a waterproof material to guarantee better watertightness.

The connection between the impervious membrane and the cutoff wall is particularly delicate. In fact, as we have seen, an attempt is made to give the curtain the greatest possible flexibility to allow it to follow the unavoidable settlements of the rock-fill, which reach their maximum in the central section of the dam but slowly decrease in degree. However, the cutoff wall is fixed and practically rigid and, therefore, the impervious membrane may eventually have to withstand important differential deformations near it.

At Salt Springs Dam, during the first few years after construction, numerous cracks appeared in the area near the junction of the face with the cutoff

*To facilitate description, "vertical joint" means a joint along the slope.

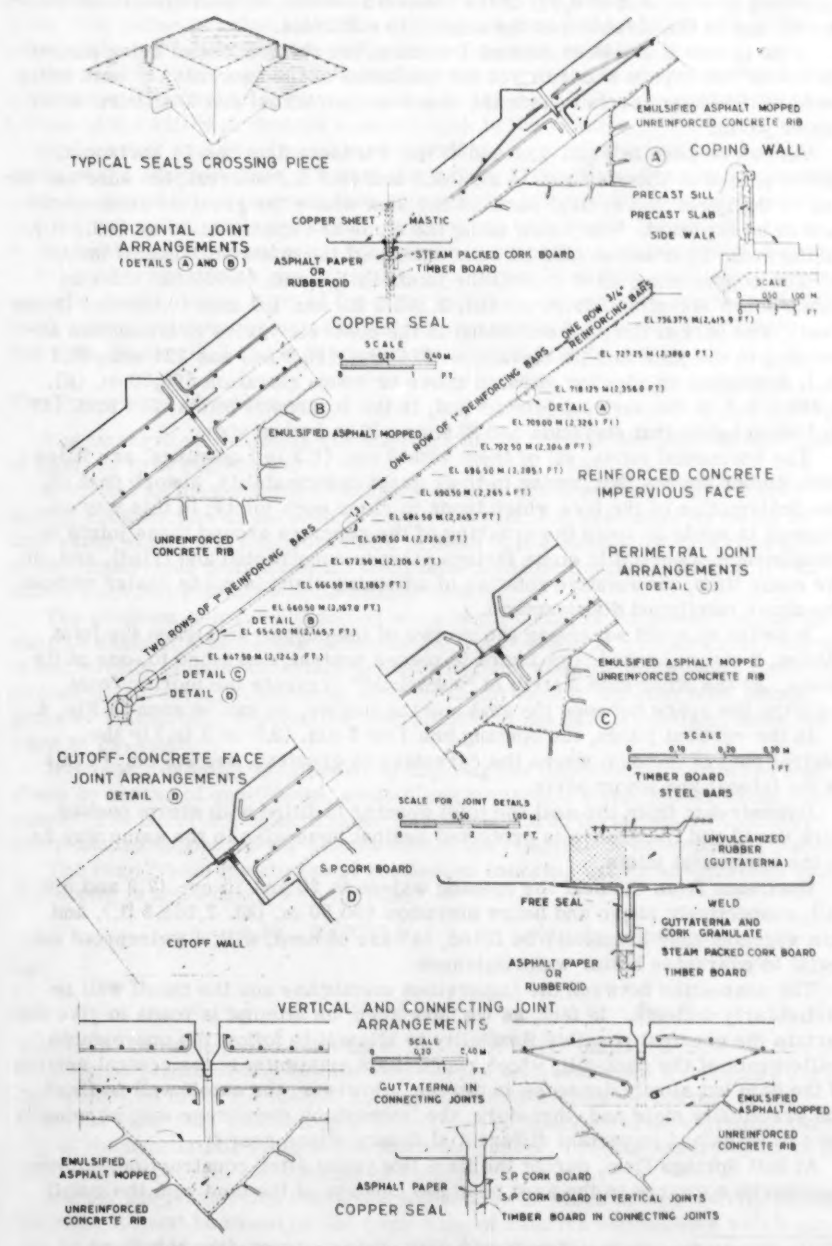
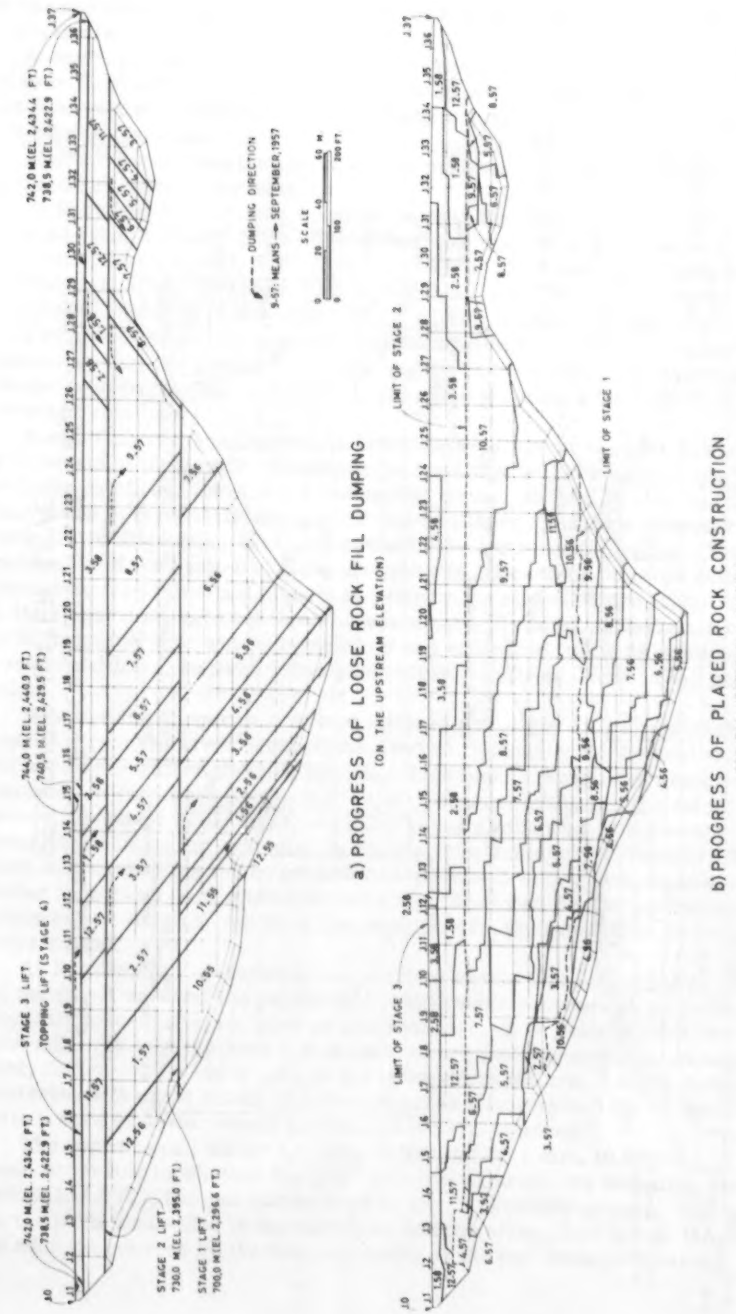


FIG. 4 - DETAILS OF THE CONCRETE FACE.



a) PROGRESS OF LOOSE ROCK FILL DUMPING
(ON THE UPSTREAM ELEVATION)

b) PROGRESS OF PLACED ROCK CONSTRUCTION
(ON THE UPSTREAM ELEVATION)

FIG. 6 - PROGRESS OF LOOSE ROCK FILL AND PLACED ROCK.

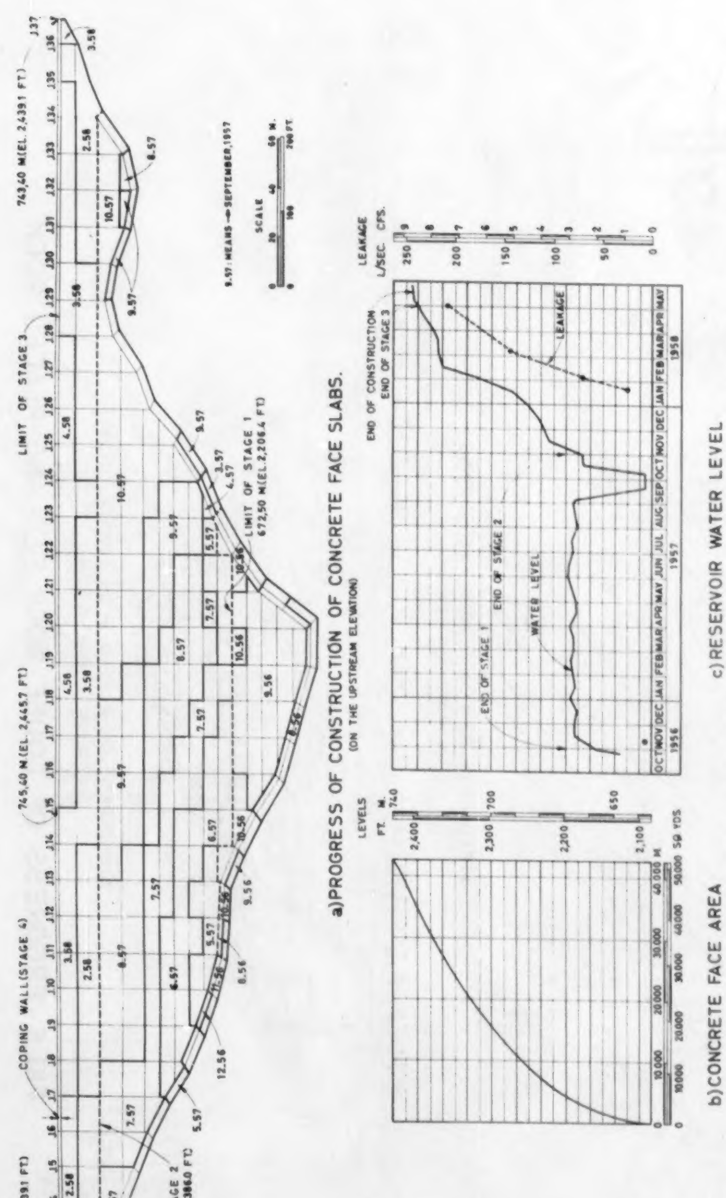


FIG. 7-PROGRESS OF CONCRETE FACE. FACE AREA. RESERVOIR WATER LEVEL.

wall and more or less parallel to the junction line. This fact was attributed to the excessive rigidity of the face in an area submitted to large differential settlements.

Accordingly, in later projects it was thought to give the curtain an even greater flexibility in that area by the use of perimetric joints parallel to the junction with the cutoff wall, thus forming a sort of articulated frame in which the face is inserted.

At Bear River Dam the use of such a perimetric joint practically stopped those cracks from appearing.

At Paradela the face was constructed with two perimetric joints: one, 4,50 m. (14.8 ft.) away from the junction with the cutoff wall, extends to about elevation 725,00 m. (El. 2,378.7 ft.); the other, at a distance of 3,00 m. (9.8 ft.) from the first, therefore 7,50 m. (24.6 ft.) away from the junction with the cutoff wall, goes up to elevation 695,00 m. (El. 2,280.3 ft.) approximately.

The existence of the large differential settlements referred to, made it necessary that the perimetric joints and the bottom joint with the cutoff wall should be able not only to open and close like the others but also to withstand stronger rotations.

Another phenomenon occurs in some rock-fill dams—and the Paradela Dam will be one of them—that influences the type of joint to be used in the junction with the cutoff wall, and in the perimetric slabs. In fact, in the case of very high dams in comparatively narrow valleys, there appears a tendency for the rock-fill to slide from the banks towards the centre of the valley. This movement of the rock-fill, although partially opposed by the arch action that the curvature of the dam intends to produce, as stated before, may carry with it the impervious membrane and cause a sort of sliding of the latter relatively to the cutoff wall which, being rigid and embedded in the foundation, can be considered fixed, as said. This same sliding can also occur in the perimetric joints.

However the copper seal is bent, it does not easily follow this sliding which can, if it reaches great proportions, lead to the tearing of the seal and the failure of the watertightening system. This problem assumes greater importance in the joint between the impervious membrane and the cutoff wall, where, the latter being fixed, the differential slidings will be greater. In the Bear River Dam, for example, an attempt was made at overcoming this drawback by interlocking the hinge slab and the cutoff wall by means of keys in order to prevent such sliding movements. However, this is equivalent to transferring the problem from the junction with the cutoff wall to the perimetric joint.

At Paradela, in preference to a solution of this type, these joints (the joint at the cutoff wall and the perimetric joints) were designed so as to follow all possible movements, isolated or simultaneous. It is believed that this aim was achieved with the help of a double watertightening system: an appropriately folded copper sheet and, in the event of its failure, a fairly deformable material with great power of adhesion to the concrete and the copper, made with a base of unvulcanised natural rubber ("Guttaterna").

The copper seal, either 1,5 mm. (0.059 in.) or 1 mm. (0.039 in.) in thickness according to whether the joint is below or above the elevation 690,50 m. (El. 2,265.5 ft.), has, as can be seen in Fig. 4, a double groove: one welded to the parts embedded in the concrete, and the other, free inside the first, is in addition covered by the free extremity of the two embedded parts.

Immediately above the copper seal, the opening of the joints is 23 cm. (9 in.) wide and it is filled with "Guttaterna" to a depth of 10 cm. (3.9 in.).

In the event of the first groove of the copper seal being ruptured owing to a serious differential sliding, the second prevents the "Guttaterna" from escaping, under the hydrostatic pressure which reaches more than 10 kg/cm2. (142.2 psi.) in the lower part of the dam, and as a result of its great plasticity, into the interior of the dam thus leaving an open way for the water.

In order to protect the "Guttaterna" from the heat when the reservoir is empty or partly filled, a board, fixed by small clamps to one of the slabs, runs along the joints leaving an opening of only 5 cm. (2 in.).

The opening of these joints below the copper seal is 3 cm. (1.2 in.) wide and it is filled, as in the horizontal joints, with timber board. As in the remaining joints, the groove of the copper seal is also protected against squeezing by "rubberoid" and steam packed cork board.

In the joint connecting the face to the cutoff wall, and in order to facilitate the movements of the former without crushing the concrete, a sheet of steam packed cork board, 8 cm. (3.1 in.) thick, is placed between the face and the cutoff wall.

The copper seals that make up the watertightening system of the joints must be continuous along all of them. Only in this way can they entirely fulfill their function.

Accordingly, it is easy to understand that the points where any two joints cross give origin to a particularly difficult problem. The continuity of the copper seals meeting at such points must be maintained, regardless of their shapes or dimensions and of the movements which they may be subject to and which may be contrary: one joint opening while the other closes.

In some American dams already constructed (Salt Springs, Dix) the seals of the vertical joints have been kept continuous while, at the meeting points, the seals of the horizontal joints have been flattened and then welded to those of the vertical joints. The different movements liable to occur there were, thus, deliberately ignored, and the copper seal was, in a manner of speaking, abandoned to its own fate.

This was done both in the crossings of the above mentioned joints and in the points where they met the cutoff wall joint or, in the case of the Bear River Dam, the perimetric joints.

At Paradela an attempt was made to solve the problem by means of the specially designed pieces shown in Figs. 4, 13 and 14. The problem is believed to have been satisfactorily solved in this way, because the continuity of the copper seals is secured and the piece has enough deformability to allow it to adapt itself to all movements to be expected.

There is a special piece for each different crossing and it must respect the direction of the joints meeting there. There are even pieces with only 3 branches, such as those used in the crossings with the joint between the concrete face and the cutoff wall.

The Crest

Above the level of the crest and on the upstream side a coping wall, 1,40 m. (4.6 ft.) high was constructed to give the necessary protection against the overtopping of the dam by waves that might originate on the surface of the reservoir.

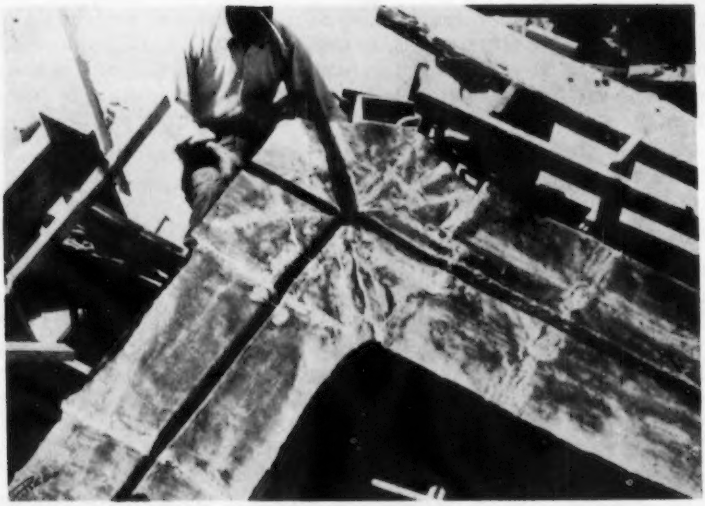


FIG. 13 — COPPER SEALS. DETAIL.
SPECIAL SEALS CROSSING PIECE.

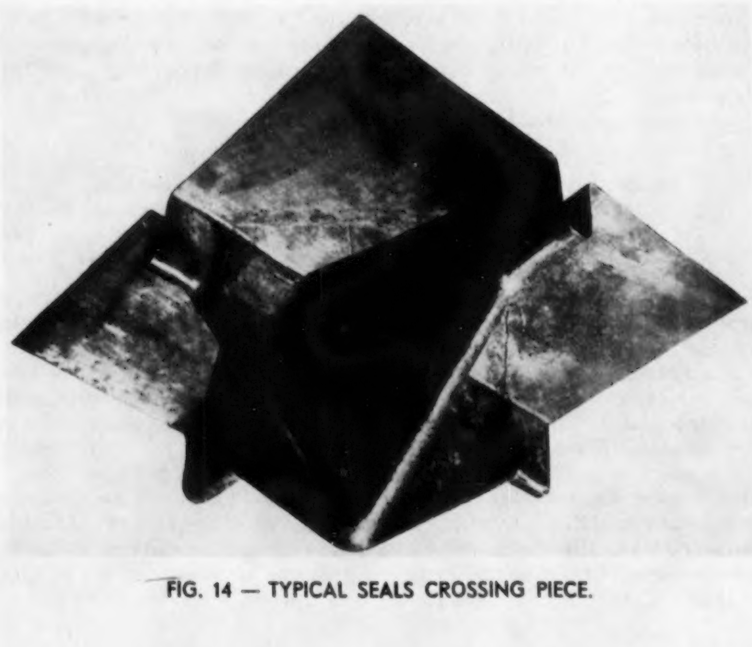


FIG. 14 — TYPICAL SEALS CROSSING PIECE.

This coping consists of pre-fabricated reinforced concrete slabs, 12 cm. (4.7 in.) thick, fitted in a groove planned for this purpose on the top of the concrete face. The length of these slabs is about 2,3 m. (7.5 ft.), and on the side ends they finish in columns, also of reinforced concrete, joined to the concrete face. Above, the coping is finished with a sill of reinforced concrete, also pre-fabricated. The allowance made in the various grooves and the tarred rope caulking around the slabs make the dam movements possible as well as the eventual readjustments that might be necessary.

General Aspects of the Construction

The Quarry

As has been already mentioned, a granite quarry was used for the construction of the dam. This quarry is about 2 km. (1.2 miles) away in a straight line from the dam, (Figs. 12 and 15) but for the transport of the rock it was necessary to make a road, 4,5 km. (2.8 miles) long with the minimum width of 9,00 m. (29.5 ft.) to provide easy transit for lorries. It must be noted that it proved worthwhile, taking European economics into consideration, to lay an asphalt surfacing so as to reduce the wear and tear of lorry tyres.

The quarry was first worked in two levels: the lower being 40 m. (131 ft.) high and the upper one of heights varying between 60 m. (197 ft.) and 40 m. (131 ft.). At a later stage the 40 m. (131 ft.) high prism of rock, remaining after the previous operations, was worked. It is the opinion of the authors that this type of quarrying with such high banks ought to be avoided owing to the considerable danger involved.

For the placing of the explosives, compressed air drills were used of a type in which the motor itself together with the "bit" goes down the hole which has a diameter of 10 cm. (4 in.), thus making long drills economical without wasting mechanical energy in transmitting it from the opening to the bottom of the hole.

The chief equipment of the quarry was as follows:

Compressors	500 HP
Drilling Machines	10 STENWICK
Shovels	3 LIMA, 3-1/2 cu. yds.
	2 LIMA, 4 cu. yds.
Bulldozers	2 CATERPILLAR D-8

The transport equipment consisted of 22 Euclids of 22 tons, fitted with hydrotarders. During some periods this equipment was reinforced by 5 Cyclops (15 tons).

It may be mentioned here that for the payment of the rockfill a price was fixed per cubic meter, measured in the quarry, including all the operations for extraction, loading, transport, unloading and sluicing. The waste was also paid for at another price per cubic meter dumped in the place destined for it. In this way it was easier to estimate the amount of work to be paid for. The volume of the waste in this dump is measured and the result reduced to rock "in situ", by applying a coefficient corresponding to the rate of volume increase. This coefficient is determined by weighing the material taken from various test pits made in the dump and previous knowledge of the density of the rock.

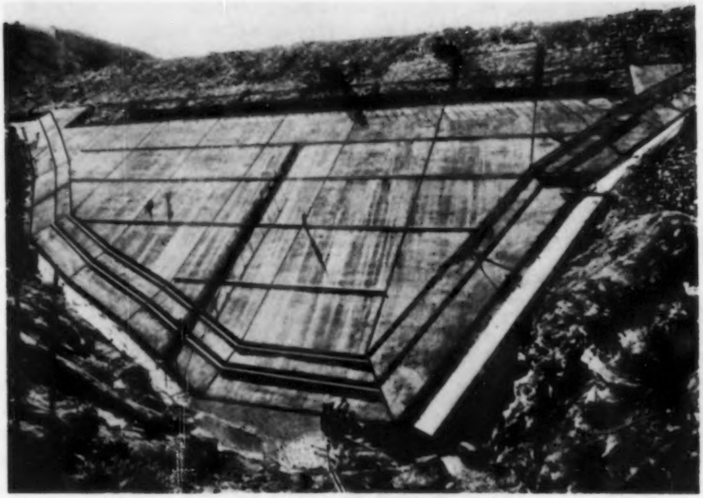


FIG. 11 — CONCRETE FACE (STAGE 1).

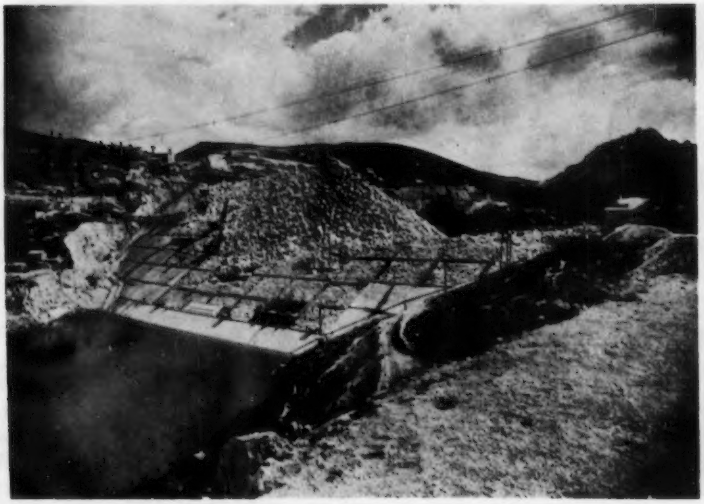


FIG. 12 — CONSTRUCTION OPERATIONS FOR STAGE 2.
(NOTICE THE QUARRY IN THE RIGHT BACK-GROUND)

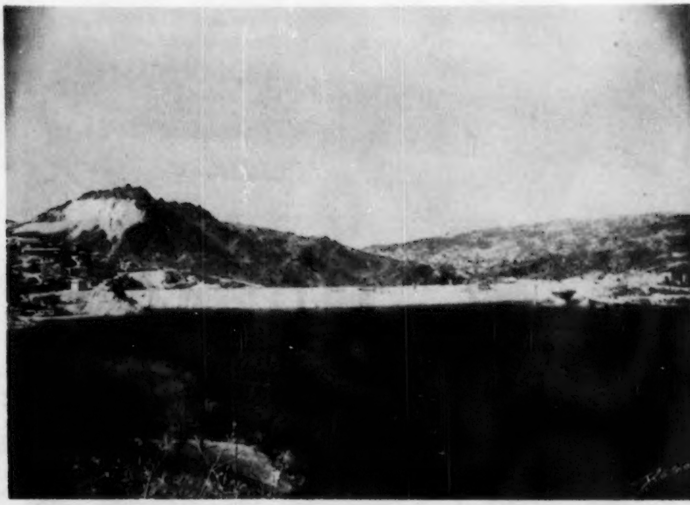


FIG. 15 — GENERAL VIEW LOOKING DOWNSTREAM.
(AT THE END OF STAGE 3)

NOTICE THE GLORY-HOLE SPILLWAY, THE AIR SHAFT AND THE QUARRY.

FIG. 16 — GENERAL VIEW LOOKING UPSTREAM.
(STAGE 3 LIFT FINISHED)

NOTICE ON THE RIGHT BANK THE DEPOSIT OF FINES REMOVED FROM STAGE 1 LIFT, THE PORTAL OF THE SPILLWAY TUNNEL AND THE OUTLET BUTTERFLY VALVE.

In this way the owner will be able to take action without prejudicing the contractor.

Of the total volume of rock extracted, about 20% was waste or considered as such. This percentage exceeded the original estimate and was due not only to geological accidents in the quarry area but also to the rejection of the fines resulting from the blasting of the rock.

This last problem—to separate the fines which are inevitable in the quarrying and to estimate if the percentage they represent is higher than that allowed by the specifications—seems to be the hardest to solve owing to the countless difficulties raised by the continuous supply of material requiring simultaneously such a great number of workmen and so much equipment. The authors are of the opinion that only training and professional consciousness on the part of the excavator operators together with careful supervision, both by those directing the work and the contractor, can solve this problem owing to the difficulty of controlling the way the specifications are being carried out.

The production peak of the quarry with the equipment described reached 10000 tons per 20 hour working day.

In the dumping of the loose rock-fill the only point worthy of note seems to be that the sluicing water used at the Paradela Dam during the dumping was equivalent in volume to four times the volume of the dumped rock, measured in the dam.

To make the sluicing operations easier the monitors were placed at the ends of overhanging booms balanced by a counterweight, the whole forming an easily movable structure on wheels. All movements of the monitor around its vertical and horizontal axes could be remote-controlled.

The arrangements described made it possible to sluice straight on to the dumping from the trucks and this proved to be the most efficient way of removing the fines not separated in the quarry.

As will be described later, very high lifts were employed from relatively narrow tracks about 20 m. (65.6 ft.) wide. This procedure made it possible to make successive partial fillings of the reservoir and resulted in a great compaction of the rock-fill.

Not only did the rocks, when falling from a great height, break their sharp points in the successive impacts but tremendous vibration energy was communicated to the whole when there were extensive slides of rock-fill. These slidings arose from the fact that unstable slopes had been formed in the dumping slope owing to the successive accumulation of piled up rocks. When a dump broke the equilibrium, large masses were set in motion and caused tremendous trepidation, estimated to last about two minutes and sometimes more.

The disadvantage of dumping large volumes of rock-fill from narrow tracks is that of the accumulation of fines that segregate at the very moment the trucks are unloaded.

To get an idea of the depth of the area it would be necessary to take away because it did not guarantee good contact between big rocks without interposing fines, experimental test excavations were made to about 6 m. (19.7 ft.) but concrete conclusions could not really be drawn as observation was very difficult. However, at that depth there seemed to be a greater percentage of big rocks. It was then decided to take away all the fines of a layer 6 m. (19.7 ft.) in height on the upstream side and 2 m. (6.6 ft.) on the downstream side, assuming that it would be preferable to achieve homogeneity in the quality of the dumped rock-fill in order to avoid differential settlements harmful to the concrete face.

This deep treatment was only carried out on the first stage lift at elevation 700 m. (El. 2,297.7 ft.). In the other lifts, as the elevations were greater, a lighter treatment was carried out, the transit layer for the lorries being removed and finally, by monitor jet treatment on the rock-fill, the big rocks on which the later dumpings from other lifts would rest were in this way left exposed.

The Placed Rock

The placed rock layer was constructed with the help of 8 cranes working simultaneously at different levels, and the adequate rock was supplied by dumping from the construction tracks on the lifts (Figs. 9 and 10). The rock utilised was always the largest available that could be handled by the equipment installed; the voids between large rocks were filled up with hand-placed rocks up to 25 kgs. (55 lbs.) in weight. The remaining voids were filled with 5 cm. (2 in.) hard crushed granite.

One problem that required attention was the accumulation of fines in the area of contact between the placed and the loose rock caused by the fines accumulated on the construction lifts (Fig. 16) being carried down when the lorries dumped their supply of rock for the construction of the placed rock layer. With the use of monitors, and even by hand excavation, those fines were removed in order to uncover the large rocks of the loose rock-fill against which the placed rock rested. The supply of rock to each place of work was not without some difficulties chiefly in the areas furthest from the lifts because the rocks reached great velocity and were in the risk of falling along the upstream face of the placed rock or on the part of the impervious face already concreted. With the previous placing of large blocks forming a retaining wall, or by constructing strong timber defences, this problem was overcome and very few large stones went beyond this fixed limit.

It should be further noted that the joints between the rocks on the upstream face of the placed rock were filled with poor cement mortar to prevent the laitance of the concrete of the impervious membrane from escaping during placing.

The Impervious Membrane

The concreting of the face (Figs. 11 and 12) did not present any special problems. The concrete used contained 300 kgs. per cubic meter (505 lbs. cu. yd.), and was made of granite aggregates from the quarry, crushed in an "Almacoa" installation, and mixed in a "Winget" tower with two 3 cu. yds. concrete mixers. The concrete was transported by two radial cable cranes of 6 tons each with two movable and one fixed towers.

The forms of the upstream face were made of sliding panels covering the distance between vertical joints and having about 1,50 m. (4.9 ft.) along the slope. They were supported on guides fitted in the ribs under the vertical joints and on steel beams resting above and below the horizontal joints that limited the slabs being concreted. Manual cable winches pulled up the forms in successive steps, 0,70 m. (2.3 ft.) at a time.

As can be seen in Fig. 4 a step was left in the concrete along the horizontal joints of the membrane. Besides facilitating the access for inspecting or repairing the face, this feature also gave sufficient head to the concrete "liquified" by vibration, for it to get under the sides of the copper seal. Air could escape through little openings made in the forms of the joint, in the part below

the groove of the copper seal. To allow the escape of the air which was still retained between the side and the groove of the copper seal, because of its position, some rubber tubes were left with one end in the space to be filled with concrete. When the laitance began flowing from these tubes which were placed at intervals of about 1,5 m. (4.9 ft.), this space was considered to be adequately filled.

As the definite figures are not yet available, it can only be said that an estimate, based on measurements made up to the present over great surfaces of the face, indicates a total consumption, in the face, of a theoretical concrete volume of 24.900 m3. (32,500 cu. yds.), to which corresponds 7.900 m3. (10,300 cu. yds.) of concrete beyond the theoretical lines.

The total weight of copper used in the construction, including the seals in the cutoff wall comes to 95 tons.

Construction Schedule

The construction schedule of the Paradela Dam was organised with a view to the possibility of production of energy even in the course of construction.

Accordingly, the work was divided into four stages, as follows:

1st Stage - Dumping of loose rock-fill from a lift 20 m. (65.6 ft.) wide at the elevation 700,00 m. (El. 2,296.6 ft.)
- Construction of the placed rock to elevation 680,00 m. (El. 2,231.0 ft.) (Fig. 9)
- Construction of the reinforced concrete face to elevation 672,50 m. (El. 2,206.4 ft.) (Fig. 11)

During this stage a provisional spillway was provided (Fig. 5) using the same tunnel as the final spillway. Also, later in this stage, the storage capacity of the reservoir was increased by raising the spillway lip by 3,00 m. (9.8 ft.) and by making a provisional timber face up to elevation 678,50 m. (El. 2,226.0 ft.).

2nd Stage - Dumping of loose rock-fill from a lift 16 m. (52.5 ft.) wide at elevation 730,00 m. (El. 2,395.0 ft.).
- Construction of placed rock and concrete face up to the same elevation. (Fig. 12)

During this stage the final spillway (Fig. 5) was used, some openings having been left in the funnel shape structure with crest at elevation 725,50 m. (El. 2,380.2 ft.).

3rd Stage - Dumping of loose rock-fill from a lift with the theoretical width of the dam at elevation 738,50 m. (El. 2,422.9 ft.), its level varying between this elevation and 740,50 m. (El. 2,429.5 ft.) so as to give the extra height of 2,00 m. (6.6 ft.) designed for the profile across the valley.
- Construction of loose rock-fill to the elevation of the construction crest.
- Construction of the concrete face to an elevation 50 cm. (19.7 in.) below the elevation of the construction crest.

During this stage the same spillway was used as in the 2nd stage.

4th and Final Stage - Dumping from the construction crest of the loose rock-fill prism over the previous stage rock-fill.

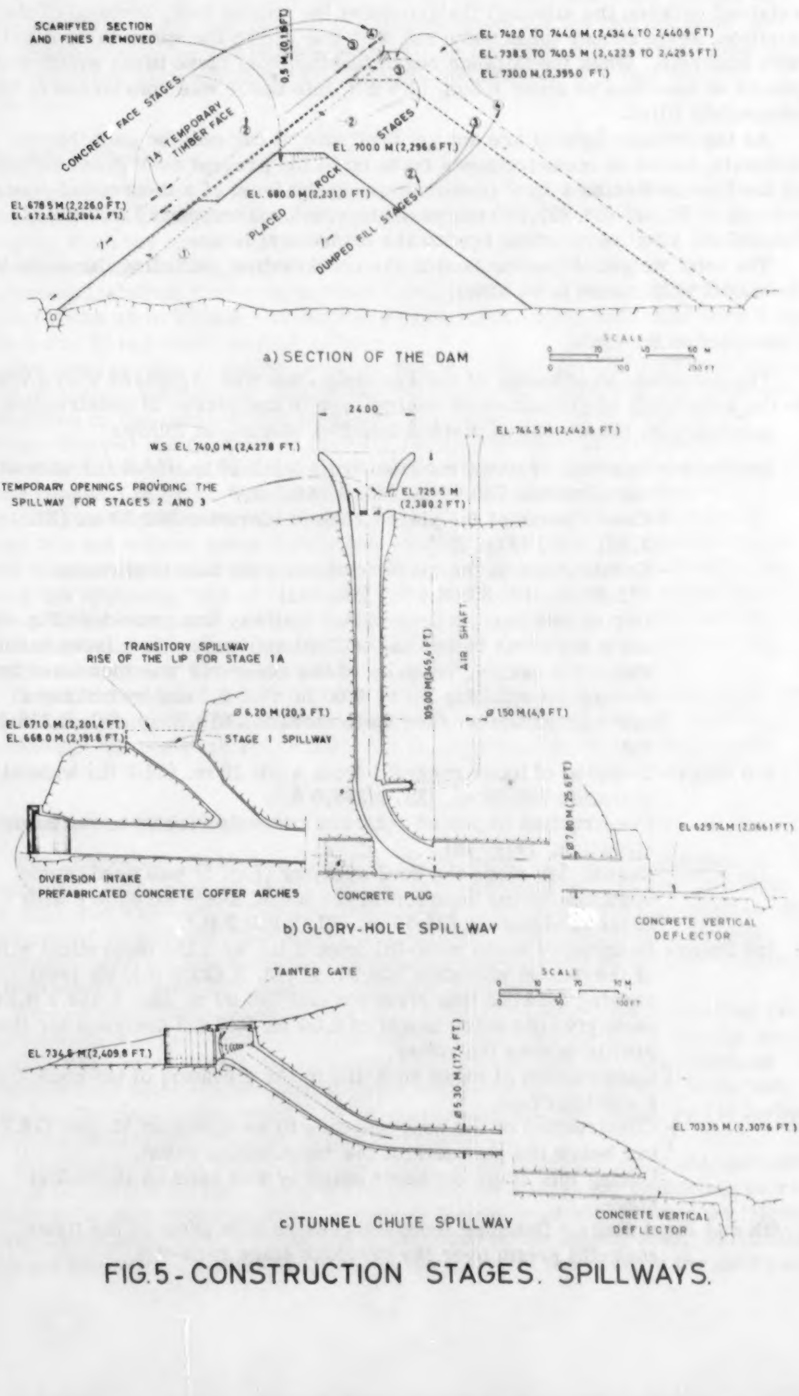


FIG. 5 - CONSTRUCTION STAGES. SPILLWAYS.

- Construction of the ridge of the concrete face slabs and of the coping wall.

The openings in the spillway structure were closed; henceforth flood control was done by the glory hole spillway (Figs. 5 and 15), with its crest at the final elevation 740,00 m. (El. 2,427.8 ft.), and by a chute spillway fitted with a radial gate and having its crest at elevation 734,50 m. (El. 2,409.8 ft.).

Energy Output

Besides the value of the reserve represented by the successive storages which were being built up in each different construction stage, the following amounts of energy were produced with the water of the Paradela reservoir in Venda Nova power house:

	Output*	Storage
Utilizing the 1st stage	88.0×10^6 kWh	Run-of-river
Utilizing the 2nd stage	62.2×10^6 "	110×10^6 kWh
Utilizing the 3rd stage	0.5×10^6 "	115×10^6 "

*Only with Paradela head.

Behaviour of the Dam

Fig. 8 shows some results of observations carried out with a view to determining the movements of the concrete face, and which can be related to the data on the growth of the dam given in Figs. 6 and 7.

Seeing that the dam has been loaded for a very short time and that it has not yet been unloaded, thus allowing further measurements, it is obvious that the elements now given have a somewhat reduced value. Therefore, the publication of further observations to be made, and which it is hoped will be a useful contribution to the greater knowledge of this type of dam, must be left for some future occasion.

However, we would like to point out that the total leakage measured with the level of the reservoir elevation 730,15 m. (El. 2,395.5 ft.) has only reached 209,5 lts/sec. (7.4 cfs.) for a concrete face area, not yet rejoined, of about 34.800 m2. (41,600 sq. yds.).

ACKNOWLEDGEMENTS

All the permanent civil engineering works for the construction of the dam were carried out by the contractors, "Sociedade de Empreitadas de Obras Públicas, Ld[a]. (SEOP)", of Oporto, Portugal.

The grouting works were conducted by "Sondagens Ródio, Ld[a]." of Lisbon, Portugal.

The steel linings, the valves and the gates were supplied and erected by "Sociedades Reunidas de Fabricações Metálicas, Ld[a]. (SOREFAME)", of Amadora, Portugal.

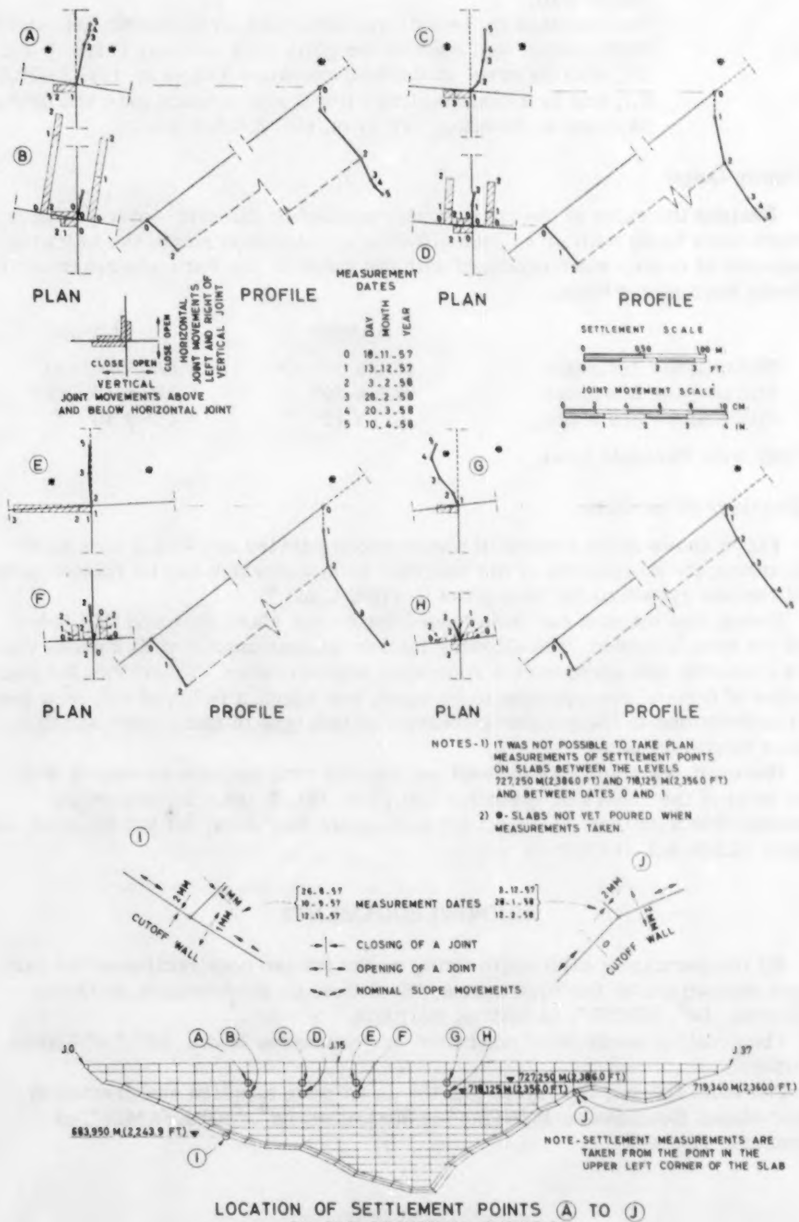


FIG. 8 - PROGRESSIVE FACE MOVEMENTS

Paper 1748 August, 1958 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: THE PARADELA DAM—FOUNDATION TREATMENT

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(Proc. Paper 1748)

FOREWORD

This paper is one of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

ABSTRACT

Foundation conditions at the site of the Paradela Dam in Portugal and the various problems they presented are described. The paper discusses the arrangement of grout curtains and other treatment carried out to suit the geological conditions, and presents performance data on seepage control.

The site of the Paradela Rockfill Dam presented various foundation problems that were resolved by an arrangement of grout curtains and other treatment to suit the geological conditions. The Paradela Concrete Face Rockfill Dam has been completely described by Messrs. Fernandes, de Oliveria and Porto in their paper (Proc. Paper 1747) that forms part of this Symposium.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1748 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

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Foundation conditions were explored in 19 test drillings totaling 668.54 meters as well as by means of 21 adits the total length of which amounts to 841.20 meters. Most of the drillings are inclined as site surface investigations disclosed that rock joints and faults are approximately in the vertical.

The rock encountered at Paradela Dam is a granite showing decomposition of hypogean character. Owing to marked weathering, one of the exploratory tunnels did not find reasonably compact rock even at a depth of 50 m.

Systematic water pressure tests were carried out in all boreholes, in stages 5 m in length, at two or three pressures, i.e. 2, 5, and 10 kg per sq. cm. on the average. By comparing the rate of water absorption at different pressures, it could be ascertained whether the water flow in the subsoil is turbulent or laminar. In the first case, water absorption is proportional to the square root of the hydraulic head, whereas in the second case the water flow, in accordance with Darcy's formula, is in direct relationship with the head. Tests results fully confirmed the general indications obtained from the cores: the permeability of this highly weathered granite is in the main similar to that of sandy material.

The granite at the dam site is intersected by several rather large fissures or faults. At the time when the corresponding tectonic movement took place, the granite along the faults was crushed into a pulp rich in clay and sand. Further decomposition and weathering utterly transformed this crushed material so that these faults offer now a risk of leakage.

The exact location of the dam was decided upon so as to make it possible to establish the cutoff in a direction coincident with that of the best granite zone. It was, of course, impossible to avoid the faults and seams orientated from upstream towards downstream. In all points were these fissures cross the cutoff, excavating was continued either to a depth where relatively less disintegrated material was found or to a maximum depth of 15 m, from the floor of the inspection gallery within the cutoff wall. At the same time, faults showing insufficient compacity were excavated downstream from the cutoff wall; the extent of this operation is shown in Fig. 2.

A rock-fill dam offers two peculiar features as far as sealing of the permeable foundation soil below the cutoff wall is concerned:

(1) The full hydraulic head from the reservoir at capacity must be met by a very small zone of soil or rock around the contact surface of the cutoff wall.

(2) It is difficult to detect and to stop grout leakage at the base of the rock-fill.

Owing to these two peculiar conditions, it was necessary to supplement the main grout curtain by several means. Two short auxiliary curtains were therefore established upstream and downstream (see Fig. 1), thus providing more extensive direct grouting of the rock in contact with the cutoff wall. Since these curtains were executed first, they created better conditions for grouting the main curtain by reducing the likelihood of grout leakage. Furthermore, the base of the placed rock, between the rock-fill and the concrete facing of the dam, was laid in cement mortar, thus preventing grout leaking in the close neighbourhood of the cutoff.

Timing of all grouting operations was scheduled in accordance with the progress of the dam structure, as deformations of the foundation under the weight of the rock-fill was to be accounted with. Grouting work began at the upstream auxiliary curtain; first, only at points where the rock-fill had

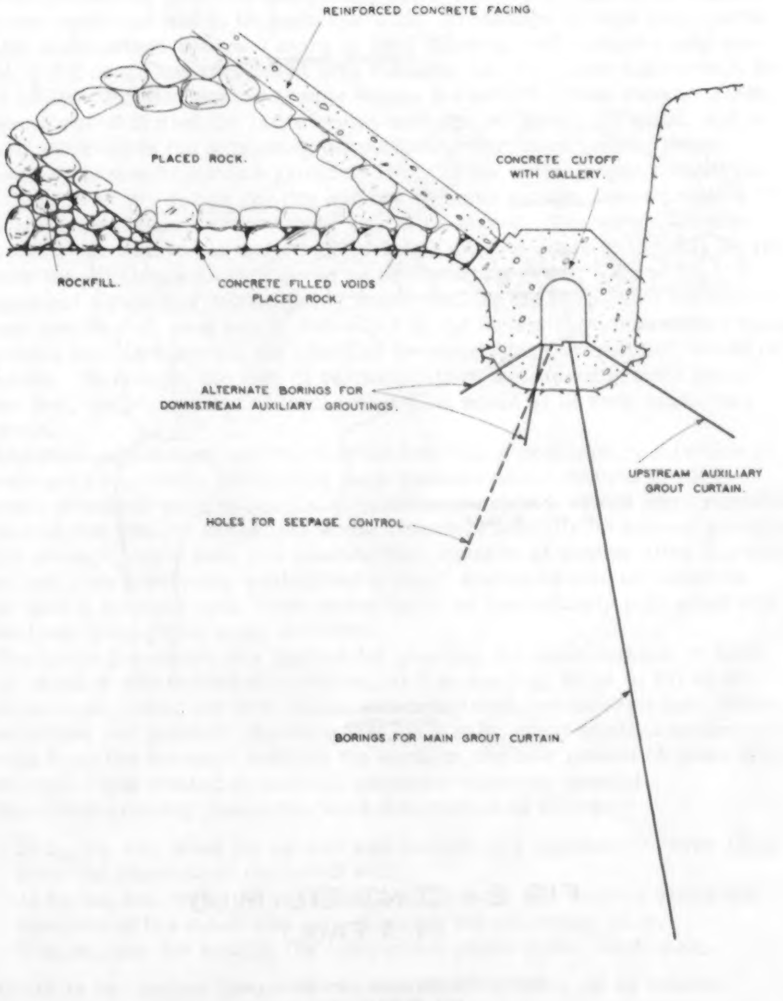


FIG. 1. - GROUT CURTAINS.

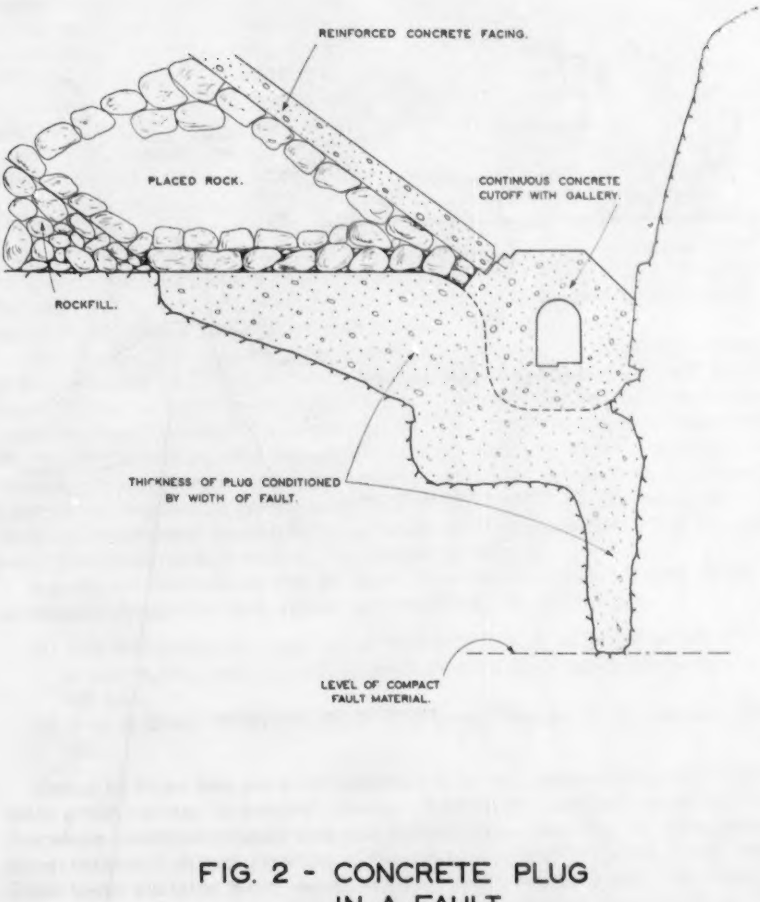


FIG. 2 - CONCRETE PLUG IN A FAULT.

BORINGS FOR PRESSURE GROUTING ARE NOT REPRESENTED.

already reached elevation 700. In doing so, it could be assumed that no further movements would take place in the foundation, with the exception of those resulting from the filling of the reservoir.

Grouting of the downstream auxiliary curtain was second on the time schedule. Once it was completed, the main curtain was grouted and, finally, with water level in the reservoir already rising, the contraction joints between the elements of the cutoff wall were sealed.

When projecting grouting operations, two kinds of possible water losses from the reservoir had to be reckoned with: (i) leakage through rock joints several millimeters wide and more or less filled up with coarse sandy material, (ii) through fissures filled with material like fine sand and through the mass of disintegrated porous granite whose permeability was known. First, it was proposed to seal the large seams with cement grout, as usual, and to remedy afterwards the subsisting permeability of rock exhibiting minor porosity by means of silicate grouting. But, as the entire upper zone of the reservoir is empty during the dry and hot summer season, decomposition of the silicate as a result of its drying was to be feared. Therefore, another procedure was arrived at and given a trial in a large-scale field test: by improving the efficiency of very finely grained cement grout, a three-dimensional network of sealed joints would enclose the permeable masses of porous granite and, as a result, the effect of the subsisting permeability would be granite and, as a result, the effect of the subsisting permeability would be harmless. Moreover, the rate of permeability of weathered granite being rather low, the occurrence of local water flow would be of very small importance.

Experienced grouting engineers know well that a preliminary injection of silicate gel succeeds in lubricating such fissures where cement grout is scarcely admitted when no special procedure is applied. Field tests repeatedly showed that granite zones that would absorb practically no cement grout in a first attempt, could take in a considerable quantity of cement after the boreholes had been previously washed and a small amount of silicate injected. After such a pretreatment, these zones could be immediately regrouted with cement and through the same drillings.

The above procedure was applied for grouting the main curtain: a first range of holes was drilled and grouted, at 5 m spacing, so as to fill up the main fissures. Once the first range was completed, two intermediate holes were drilled and grouted. Knowing that, as a rule, grout shows a tendency to flow up from the borehole towards the surface, the next groups of grout holes were drilled and treated in downhill sequence whenever possible.

Maximum grouting pressures were determined as follows:

- 20 kg/sq. cm. when the packer was located at a distance of /over 10 m from the concrete of the cutoff wall,
- 10 kg/sq. cm. when the packer of the borehole was located within the concrete of the cutoff wall or a distance not exceeding 10 m,
- 5 kg/sq. cm. for sealing the contraction joints in the cutoff wall.

Depth to be reached by the three grout curtains were set as follows:

- Upstream auxiliary curtain: 5 m down into granite;
- Downstream auxiliary curtain: 2 m;
- Main grout curtain: Drillings were carried out down till they met practically impermeable rock, i.e. down to a 5 m stage whose absorption—checked in a 10 minute test after the water flow had become stabilized—

appeared to be less than one liter per minute and per meter, the pressure applied being 10 kg/sq. cm. A minimum depth of 20 m was ordered.

Amount of grout used, per meter of cutoff wall:

- Cement 1564 kg, chemicals 169 kg, in the three curtains.
- Total drilling: 18.80 m per meter of cutoff wall.

Whenever a water loss from the reservoir is observed in a structure of the type of Paradela Dam, it is impossible to locate the leaks as neither the concrete facing nor the soil area in the close neighbourhood of the cutoff wall are free for inspection. But, considering the danger which the occurence of piping may involve, it is imperative to provide for its immediate location so that measures to prevent a disaster may be taken.

At Paradela Dam this problem was overcome by creating means for a close checking of the behaviour of the grout curtains. An extraordinary large number of seepage control holes were bored after completion of the grouting works (see Figs. 1 and 4). Thanks to these safeguard measures, should piping occur, it would be detected by the unusual inflow into the gallery through the observation borehole controlling the defective zone within the main grout curtain.

The discharges collected in each of these observation holes is recorded by individual graphs. The measurements thus obtained evidence whether the rate of discharge and the head keep a linear character or not. From the readings of most of these graphs, it could be deduced that, as anticipated, the corresponding flow was laminar, i.e. in accordance with the basis of Darcy's formula. Should a curve depart from the linear character as a result of increased discharge, this sign would indicate unsatisfactory conditions and additional grouting would be carried out without delay in the zone controlled by the respective hole. Fortunately nothing of the kind happened.

The graphs furnished also some unexpected and interesting information. Winter rainfall upon the disintegrated granite caused considerable subsurface water flow down the adjoining hills. After a period of several days of dry weather, while the water level in the reservoir was still rising, the discharge from most of the holes for seepage control decreased as indicated in Fig. 5.

The total water losses measured at the downstream heel of the dam amounted to 209.5 litres per second, with the reservoir at elevation 730.16 meters. The total inflow into the cutoff wall gallery, through the holes for seepage control was of 1.4 litres per second at the same time.

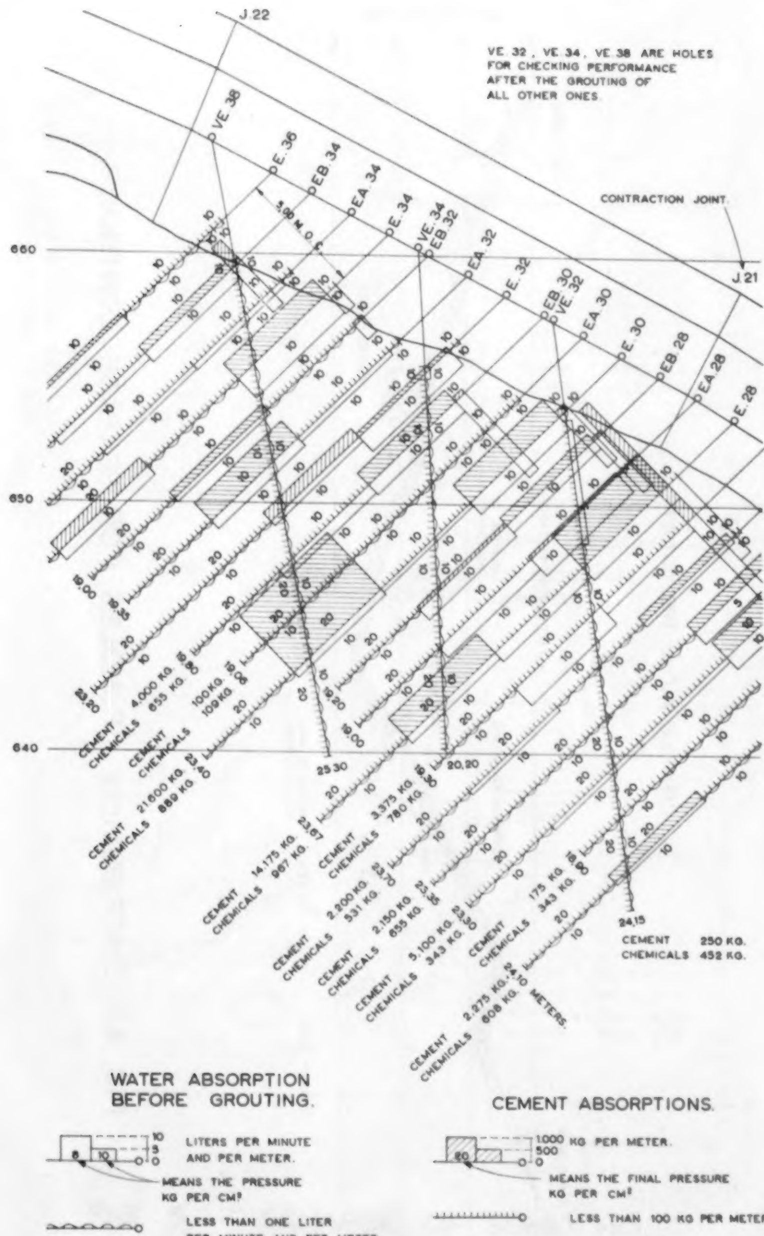


FIG. 3. - FRAGMENT OF MAIN GROUT CURTAIN.

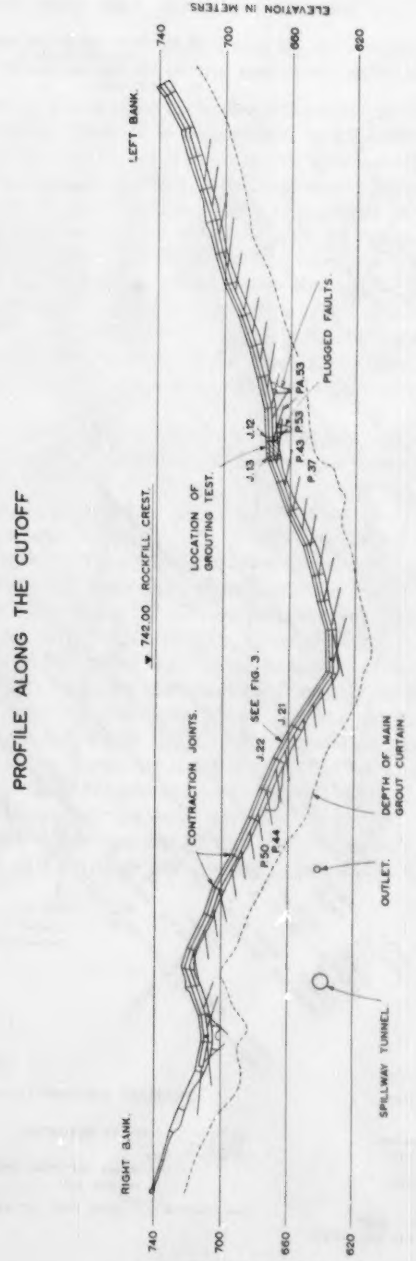
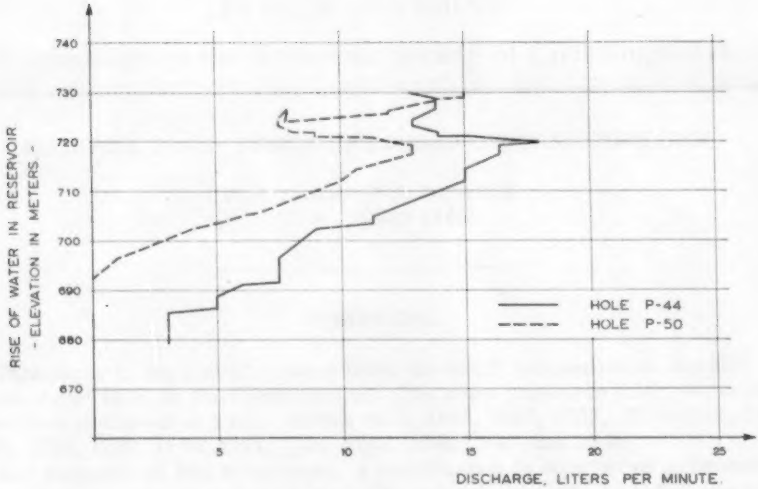


FIG. 4 - DISTRIBUTION OF HOLES FOR SEEPAGE CONTROL.



OF ALL HOLES FOR SEEPAGE CONTROL, P-44 SHOWED THE HIGHEST DISCHARGE.

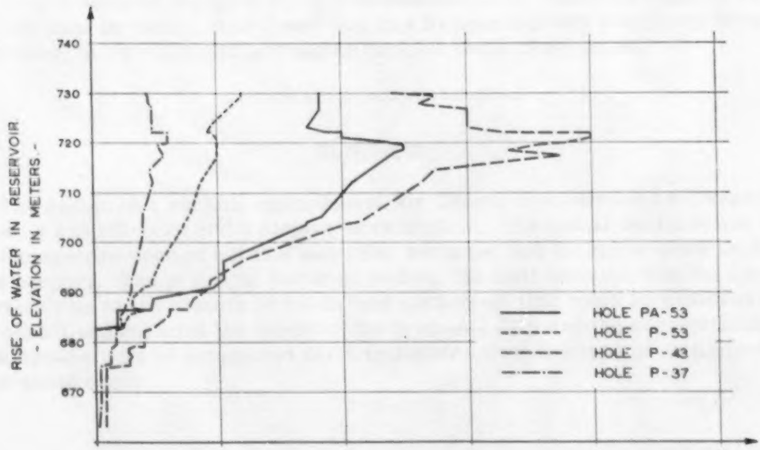


FIG. 5. - DISCHARGE OF SOME HOLES FOR SEEPAGE CONTROL.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ROCKFILL DAMS: DESIGN OF COUGAR CENTRAL CORE DAM

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(Proc. Paper 1749)

FOREWORD

This paper is the last of a group from the ASCE Symposium on Rockfill Dams, June, 1958, at Portland, Oregon. The other papers in this Symposium have been published as Proc. Papers 1671, 1683, 1687, 1733, 1734, 1735, 1736, 1737, 1738, 1739, 1740, 1741, 1744, 1745, 1746, 1747, and 1748.

For purposes of this Symposium, a rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

The objective of the Symposium is to assemble experience data on the higher rockfill dams of all types along with discussion by engineers engaged on rockfill dam projects. It is hoped that this Symposium will contribute toward improved, more economic and higher rockfill dams of all types.

SYNOPSIS

The design of a rockfill embankment for Cougar Dam involved a choice between a central-core and a sloping-core section. The initial decision was for a sloping-core section but this was later reversed and a central-core section was adopted. The principal factor in making the final decision was the uncertainty as to the amount of water load settlement that could be expected in a rockfill embankment the height of the proposed dam and the susceptibility of a sloping core of compacted earth to rupture from the type of settlement that would occur.

Note: Discussion open until January 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1749 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 4, August, 1958.

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INTRODUCTION

Cougar Dam and Reservoir will be a unit in the general comprehensive plan for flood control, navigation, and other purposes in the Willamette River Basin in northwestern Oregon which was authorized by Congress in the Flood Control Act of 1938 (Public Law 761, 75th Congress, 3rd Session). The primary function of the project will be flood control. Other functions will include power generation and storing water for release during periods of low natural flow to aid irrigation, navigation, and pollution abatement on the lower stretches of the river system.

The dam site is located on the South Fork of the McKenzie River, 4.4 miles above the confluence of that stream with the McKenzie River and 42 air miles east of Eugene, Oregon. (Figs. 4 and 5) The dam, as now planned, will be 1580 feet long at the crest and 445 feet high above the foundation with the axis arched upstream. It will control the runoff from 210 square miles of mountainous and timbered land on the western slopes of the Cascade Mountains, practically all of which is within the Willamette National Forest. The reservoir will cover 1280 acres at maximum pool and will have a storage capacity of 219,000 acre-feet, of which 155,000 acre-feet will be available for flood control storage.

Appurtenant structures will include a concrete spillway on the right abutment, power and regulating outlet tunnels through the left abutment with a common intake structure, and a powerhouse near the downstream toe of the dam. The spillway discharge will be controlled by two tainter gates, each 40 feet wide by 43.5 feet high. The power tunnel will be steel-lined, 10.5 feet in diameter and 1200 feet long, and the regulating outlet tunnel will be concrete-lined, 13.5 feet in diameter and 1000 feet long. The initial power installation will be two 12,500 kilowatt units with provision for expansion to accommodate an additional unit if future power demand justifies a peaking plant.

The diversion tunnel, a horseshoe-shaped unlined tunnel 19.5 feet in diameter and 1800 feet long, was completed in 1957 and approximately 1,400,000 cubic yards of talus overburden was removed from the abutment slopes under a separate stripping contract during that year. A road along the west side of the reservoir, to replace the existing Forest Service road in the valley, is under construction and will be completed in the late fall of 1958. It is planned to award the contract for construction of the dam and appurtenant structures in the early spring of 1959 with completion scheduled for the fall of 1961.

Geology and Foundation Conditions

The dam site is located in the Western Cascades, a geologic province of early and middle Tertiary pyroclastics, lava flows, and contemporaneous intrusives. The oldest rock in the reservoir area is a series of bedded pyroclastics which form the valley floor at the dam site. Following the accumulation of these pyroclastics there was a gentle regional warping which tilted the area toward the east. After several cycles of vigorous erosion followed by renewed volcanic activity which filled the eroded valleys with lava flows and additional pyroclastic deposits, the formation of the present Cascade mountain range blocked the drainage to the east, resulting in extensive valley filling with alluvial material and starting the drainage pattern to the west which exists today.

Fig. 4 View of Damsite Looking Downstream 50-319

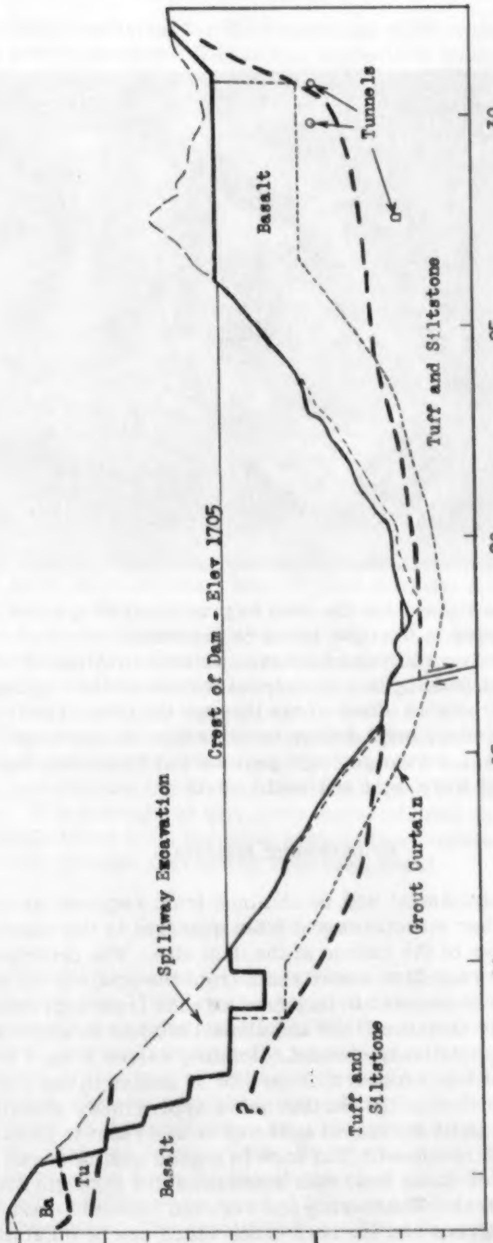
Fig. 5 View of Damsite Looking Upstream 50-324

The region is characterized by prominent sharp ridges and steep heavily timbered slopes with shear rock cliffs and broad high-level benches, which form a topography of early maturity. The valley slopes are mantled with a mixture of residual soils, slope wash and talus, and the valley floors are covered with varying depths of recent river alluvium. The river flows in a bouldery channel with many rapids and constrictions produced by boulder terrace deposits and by irregular erosion of the pyroclastic rocks and igneous intrusives. At the dam site it has cut a precipitous canyon some 700 feet deep through a basalt mass.

A geologic section of the dam site at the axis is shown in Fig. 2. The bedrock in the valley floor is a flat-lying series of alternating layers of mudstone and tuff. The results of pressure tests in core drill holes in this formation indicates that foundation leakage will not be a problem, and it is planned to limit the initial grouting in the valley floor to exploration grouting and to expand this program in areas where the grout take indicates that additional grouting is necessary. The overburden on the valley floor is a mixture of sand, gravel and boulders which varies in depth from 5 feet to 30 feet except in potholes where a depth of 70 feet has been measured. This alluvium, except for a shallow surface mantle of sand and silt, has been well consolidated at all locations where it has been explored. The surface mantle will be removed from the entire area under the embankment, but it is not planned to excavate to bedrock except under the core and possibly in an exploration trench at the downstream toe of the dam.

The sides of the canyon are formed by massive basalt outcrops which overlie the tuffaceous sediments in the valley floor and rise almost vertically from the valley slopes to heights above the crest of the dam. These basalt masses are backed by a series of tuffaceous sediments similar to those in the valley floor which outcrop above the basalt on both abutments. The contact of the basalt and the tuffs was tight and usually welded in all the drill holes in which it was encountered. The overburden on both abutments is a mixture of basalt talus and sandy silt slope wash. The major accumulation of this talus overburden is downstream from the axis on the left abutment and depths of 100 feet have been measured in drill holes in this area. All of this material will be removed from the foundation area under the embankment.

The principal joint pattern in the area consists of two nearly complementary sets of joints dipping from about 70 degrees to vertical. These joints occur in the basalt on both sides of the canyon. On the left side one set of joints is nearly parallel to the canyon wall and the wall downstream from the axis is formed by almost vertical slabs more or less separated along these joints. (Fig. 6) The rock exposed above the talus slope is sound basalt, but when the talus was removed it was found that the basalt which had been covered was badly weathered and disintegrated to depths of about 30 feet. After the talus and weathered rock was removed from in front of the vertical slabs, some sections toppled into the excavation indicating that the bases of these slabs are also disintegrated. It is believed that the disintegration resulted from the action of water trapped in the open joints and in the talus. This condition is limited to the area downstream from the axis on the left abutment (Fig. 6) but will require the extensive removal of weathered rock and the removal of some sound rock in that area. It is anticipated that more extensive grouting will be required in the abutments than in the valley floor because of the jointing in the basalt. The extent of the proposed grout curtain is shown in Fig. 2. Horizontal holes will be drilled into the left abutment along the slope of the



GEOLOGIC SECTION AT AXIS

Fig. 2

CO-318

Fig. 6 View of Left Abutment

embankment downstream from the axis to provide drainage and prevent the accumulation of water in the open joints in the basalt.

There are no known records of seismic activity originating in the vicinity of the dam site, but faulting is a prominent feature of the regional picture and some faults of appreciable offset cross through the site. These have not been considered in the embankment design because they do not appear to have suffered movement in recent geologic periods and those exposed in excavating the diversion tunnel were tight and well healed.

Embankment Materials

Rock for the embankment will be obtained from required excavation for the spillway and other structures and from quarries in the basalt outcrops which form the sides of the canyon at the dam site. The principal quarry will be developed in the rock face downstream from the spillway on the right abutment. The basalt exposed in this face extends from approximate elevation 1400 to approximate elevation 2100 and about 1800 feet downstream from the limits of the spillway tailrace channel. Jointing varies from 4 to 12 inches in the upper portion of this exposure, from 6 to 36 inches in the central portion and from 18 to 60 inches in the portion below approximate elevation 1500. A secondary quarry can be developed in the area upstream from the embankment on the left abutment. The rock is a good quality basalt, weighing 150 to 160 pounds per cubic foot, with a compressive strength above 20,000 pounds per square inch. Weathering and regional tectonics have opened the joints to varying degrees and the rock sizes which can be obtained will be largely controlled by the extent to which this has occurred. Weathering, except at shear planes and faults, is generally limited to an oxidized film or thin coating along the joints which does not impair the quality of the rock.

Locating a source of material for the impervious core in the vicinity of the dam site has been a problem. The larger deposits of suitable material occur as terraces on the upper portion of the valley slopes. Several of these were investigated with auger borings and bulldozer trenches and all were found to contain similar material. The most favorably located sources are two areas on the slopes above the right abutment, about 1.5 miles from the site by haul road. The material is a sandy silt with a liquid limit ranging from 45 to 60 and a plastic index ranging from 11 to 17. The natural moisture contents in all deposits explored were well above the optimum and the material will have to be dried before it can be compacted with the heavy pneumatic-tired rollers which will be used. The shear strength is high when the material is compacted at the moisture contents at which it can be used. Several deposits of gravelly material were explored in an effort to find a material suitable for use in the core which would be easier to work and could be worked over a longer season. The materials in the deposits explored contained sufficient fines to be impervious but they also contained cobbles and boulders in such amounts that processing would be necessary to remove them and this would be difficult because of the high moisture content and the clayey nature of the fines.

Approximately 1,400,000 cubic yards of material have been removed from the abutment slopes and disposed of in a waste area just upstream from the dam site. This material is a mixture of basalt talus and sandy silt, with varying amounts and sizes of talus rock. Much of this material appeared to be suitable for use in the core but some portions did not contain sufficient fines and all contained large rock and boulders. An effort was made to segregate the more suitable material in the disposal area and dragline trenches are being excavated so that this material may be examined more carefully and samples obtained for laboratory testing.

Material for the transition zone will be obtained from river bars and adjacent flood plain deposits in the vicinity of the dam site but it will probably be necessary to manufacture the filter materials in order to obtain the required gradation. It is anticipated that filter materials and concrete aggregates will be manufactured from the local gravels in a combined operation since no satisfactory natural aggregates have been found.

Embankment Design

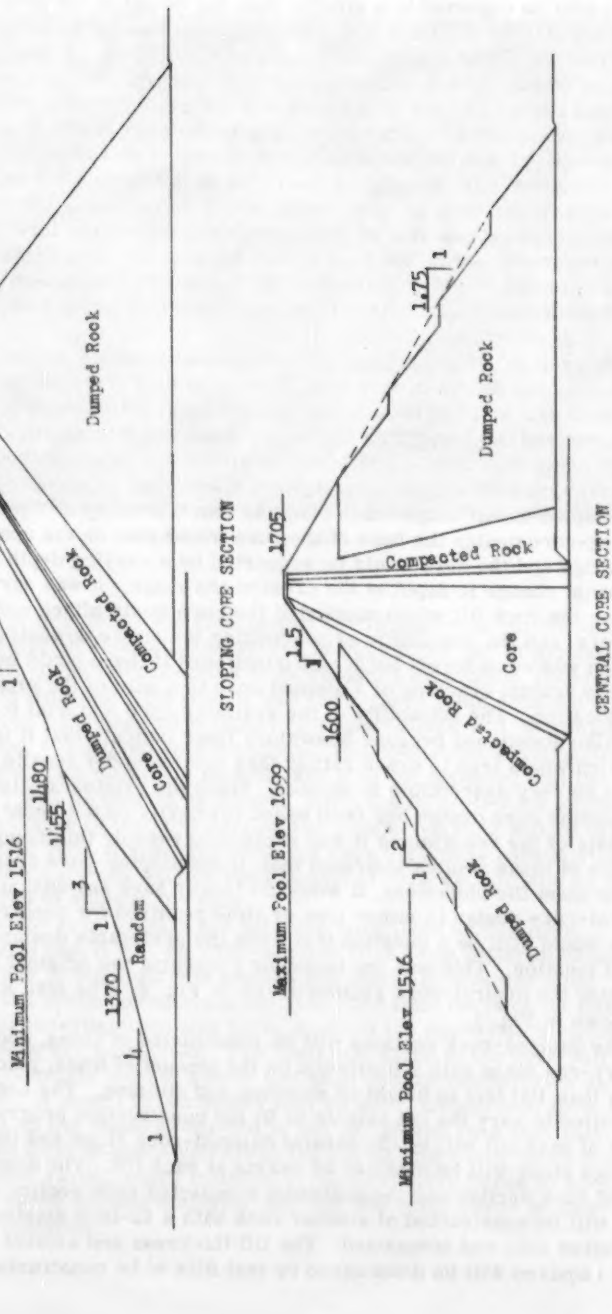
The design originally proposed for Cougar Dam in House Document 531, 81st Congress, 2nd Session (1950), was a gravel embankment with a compacted earth core, similar to the designs used for other dams in the Willamette River Basin. It was found, however, early in the advance planning investigations, that the construction of this type of embankment would require extensive borrow pits in gravel deposits near the mouth of the South Fork and that local interests would object to this excavation because it would impair the natural beauty of the area. Since an adequate quantity of rock for a rockfill embankment could be obtained from the basalt outcrops which form the canyon walls at the dam site, studies were made of the comparative costs of a gravel and a rockfill embankment. The results indicated that the total project cost would be less with a rockfill embankment than with the gravel embankment originally proposed.

Two types of rockfill embankment were considered, one with a centrally located compacted earth core, the other with a sloping compacted earth core located upstream from the axis. The central-core design would require less foundation excavation and a smaller quantity of impervious core material but the sloping-core design has the advantage that, under normal weather conditions, it could be constructed in a shorter period of time with a consequent saving in construction cost. The long rainy season in the Willamette River Basin limits the period during which impervious materials can be placed to from three to four months in normal years and may limit it to less than two months in wet years. Rock, on the other hand, can often be quarried and placed the year round at the elevation of the dam site, and for all except two or three months during the most severe winters. With a central-core design the rate at which the rockfill can be constructed will be controlled by the rate at which the core material can be placed. On gravel embankments constructed in this area it has usually been possible to continue placing gravel after the fall rains have halted the placing of core material but the length of time during which this can be done is limited, particularly during the later stages of construction, by the height to which the gravel can be carried above the core. When this height is reached it is necessary to stop all work on the embankment until the placing of core material can be resumed the following summer. With the sloping-core design the rockfill section downstream from the core, which contains the major portion of the rock in this design, can be completed independently of the core, and scheduling studies indicate that this could decrease the time required for construction by as much as one year.

The sloping core section shown in Fig. 3 was adopted because the results of the preliminary studies indicated that a dam of this design could be constructed at a substantially lower cost than one with a central core. It was planned to place the rock in the downstream dumped-rock section in one lift by dumping and sluicing from elevation 1610 and to face this with a compacted rock section to support the core. The compacted rock would be placed in 12-inch to 18-inch lifts and compacted with a 50-ton roller equipped with steel wheels.

Since the proposed dam will be appreciably higher than any rockfill dam with a sloping core previously constructed, 445 feet above the foundation compared with 325 feet for Kenney Dam, there was considerable concern about the amount of settlement that would occur when the reservoir was filled and the possible effect of this settlement on the sloping core. Detailed information was obtained on the settlement of three rockfill dams with reinforced concrete faces, Salt Springs Dam (328 feet high), and the Lower Bear River Dams (245 feet high and 150 feet high) constructed by the Pacific Gas and Electric Company in California. Essentially all of the water load settlement in these dams took place during the first complete filling of the reservoir. Two types of movement were observed in each dam, a settlement normal to the concrete facing which was maximum in the central portion of the dam about four-tenths of the height above the base, and a lateral settlement away from the abutments which was maximum at the abutments and tended to open joints and cause cracks in the upper portion of the concrete facing at or near the abutments. The lateral settlement in each case was roughly proportional to, and appeared to result from, the normal settlement.

The settlement data on these dams indicates that the magnitude of water load settlements in rockfill dams may increase at a rate greater than in direct proportion to the height of the dam. Extrapolation from the measured



Maximum Pool Elev 1699
Minimum Pool Elev 1516
1705
1610
1640
1480
1455
1370
1.5
1
2.2
1
3
1
4
1
Dumped Rock
Compacted Rock
Dumped Rock
Core
Random
SLOPING CORE SECTION
1705
1.75
1
Dumped Rock
Compacted Rock
1.5
1
Core
Compacted Rock
1600
Maximum Pool Elev 1699
2
1
Dumped Rock
Minimum Pool Elev 1516
CENTRAL CORE SECTION

Fig. 3

settlements indicates that a maximum settlement of 6 to 10 feet normal to the face might be expected in a similar dam the height of the proposed dam. It is expected that the settlement of a sloping-core rockfill dam will resemble that of a concrete-faced rockfill dam and could be greater because of the flatter slope on which the water load is applied. The probable high maximum settlement normal to the face of the core was not regarded as a problem in itself but the possibility of cracks developing in the core due to lateral settlement of the rock fill was thoroughly studied. Tension in a rock fill, such as occurs near the abutments, appears to open cracks in the rock fill and, in a sloping-core dam, would tend to open cracks in the core. It was observed that cracks normal to the axis opened at Salt Springs dam above the level of the partially filled reservoir during the first year. Arching the core in plan would not protect it if cracks developed ahead of the rising reservoir level because water would enter the cracks before there was sufficient water pressure to close them by arch action.

The problem of protecting the core against possible rupture became more serious as the design progressed. The relatively steep abutment slopes at the proposed site will be conducive to large lateral settlements in a rockfill embankment and the topography at the left abutment is such that it would be difficult to avoid high tensile and shearing stresses in a sloping core. The most favorable location for the axis at the left abutment is along the crest of a steep-sided basalt ridge which projects into the valley at this point. In a sloping-core design the base of the core would rest on the upstream slope of this ridge and the core would be supported by a varying depth of rock fill with an abrupt change in depth at the crest of the ridge. It was agreed that compacting the rock fill which supported the core would afford some protection to the core, and the possibility of compacting the entire downstream rockfill section was considered, but it was questioned if there could be any positive defense against cracking of a sloping core in a dam of the proposed height at this location. The suitability of the available core material for a sloping core was also questioned because laboratory tests indicate that it is an MH material which would tend to crack rather than deform under tensile stress and would be very susceptible to erosion. Since the original decision in favor of the sloping core design had been based primarily on a comparison of estimated costs of the two designs it was decided to restudy this comparison. The results of these studies indicated that, if the sloping-core design were modified to meet the objections, it would no longer have any advantage over the central-core design in either cost or time required for construction, and there would still be a question if it were the preferable design for the proposed location. This was the basis for reversing the original decision and adopting the central-core section shown in Fig. 3. The plan for this section is shown in Fig. 1.

The dumped-rock sections will be constructed of clean, sound basalt in quarry-run sizes with a limitation on the amount of fines, placed in lifts not more than 100 feet in height by dumping and sluicing. The contractor will be permitted to vary the lift heights to fit his construction program. The outer slope of each lift will be the natural dumped-rock slope and the required average slope will be obtained by berms at each lift. The downstream compacted rock section and the upstream compacted rock section above elevation 1600 will be constructed of similar rock with a 12-inch maximum size, placed in shallow lifts and compacted. The lift thickness and amount of compaction to be required will be determined by test fills to be constructed this summer.

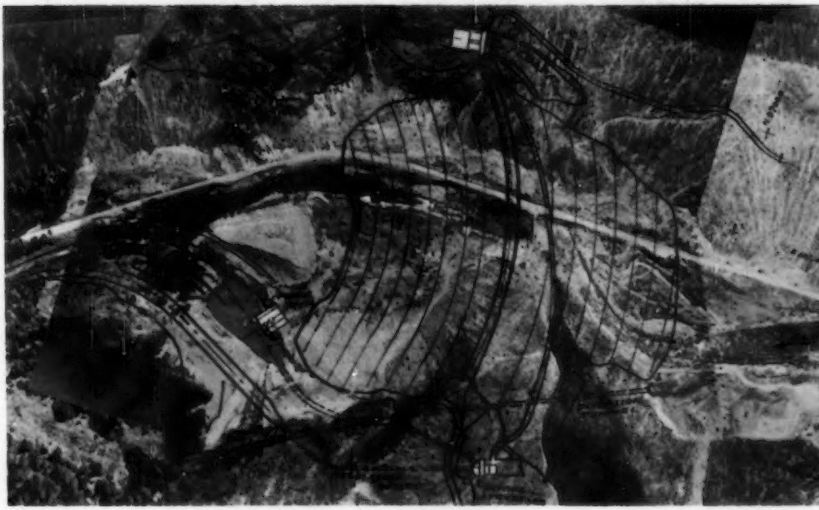


Fig. 1 Plan of Cougar Dam

The quality requirements for the rock placed in the upstream compacted rock section below elevation 1600 will be relaxed to permit the use of partially weathered basalt, sound tuffs obtained from required excavation, boulder alluvium excavated from the core trench, and the cleaner talus obtained from abutment stripping. Placing and compacting requirements will be similar to those for the other compacted rock sections. The downstream compacted rock section shown is a minimum section, and it is not expected that the downstream boundary will be exactly as shown. When the dumped-rock section is constructed in advance of the core section the compacted rock will be placed against the dumped rock slopes, and when the core section is constructed in advance of the dumped-rock section the compacted rock section will be of the thickness required to support the core.

The design of the core section has not been completed at this time. Present thinking is that it will be a two-section core, with material obtained from borrow areas in the upstream section and material obtained from abutment stripping in the downstream section. The final decision on this will not be made until investigations now being made of the materials stripped from the abutments are completed. The upstream slope of the core will be protected by a transition zone of gravel and sand and the downstream slope by a two-zone filter. A zone of rock spalls may be used downstream from the filter, depending on the results obtained in the compacted rock test fills.

The estimated total in-place volume of the embankment now being designed is 13,200,000 cubic yards, which includes 11,000,000 cubic yards of rock, 1,600,000 cubic yards of core material, and 600,000 cubic yards of filter and transition zone material.

ACKNOWLEDGEMENTS

The Board of Engineering Consultants for Cougar Dam is composed of Arthur Casagrande, M. ASCE; James P. Growdon, M. ASCE; and I. C. Steele, M. ASCE. The dam is being designed by the U. S. Army Engineer District, Portland, Oregon; Colonel Jackson Graham, District Engineer; Ben L. Peterson, Chief, Engineering Division. The writer is grateful to Mr. Peterson for his review and constructive criticism, and acknowledges his debt to Robert J. Pope, Chief, Soils Engineering Section, who supervised the embankment design, and to Lloyd L. Ruff, Chief, Geology Section, who supervised the explorations and foundation design, for their contributions to this paper.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)c, 1347 (BD1), 1348(HY4)c, 1349(SA4)c 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)c, 1378 (HW4)c, 1379(IR2), 1380(HW4), 1381(WW3)c, 1382(ST5)c, 1383(PL3)c, 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)c, 1416(PO5)c, 1417(HY5)c, 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)c, 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)c, 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)c, 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)c.

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)c, 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)c, 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)c, 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)c, 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)c, 1521(IR1)c, 1522(ST1)c, 1523(WW1)c, 1524(HW1), 1525(HW1), 1526(HW1)c, 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)c, 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)c, 1558(HY1)c, 1559(SM1)c.

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)c, 1577(PL1), 1578(PL1)c, 1579(WW2)c.

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)c, 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)c, 1615(IR2)c, 1616 (HY2)c, 1617(SU1), 1618(PO2)c, 1619(EM2)c, 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)c, 1653(WW3)c, 1654(SM2), 1655(SM2), 1656(ST3)c, 1657(SM2)c.

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)c, 1689(PO3)c, 1690(HY3)c, 1691(PL2)c.

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)c, 1722(ST4), 1723(ST4), 1724(EM3)c.

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

c. Discussion of several papers, grouped by divisions.